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Coming next month in the JOURNAL

"Re-Proportioning of Concrete Mixtures for Air Entrainment," submitted by H. J. GILKEY will appear in the February issue. Following a summary of current knowledge of air entrainment, the problems of mixture re-proportioning are presented and illustrated with unit-block solid-volume diagrams.

●

A trio of authors, W. J. McCOY, R. J. SWEITZER, and M. E. FLENTJE have joined in preparation of "Study of Concrete Pipe in Service." Originally presented at the 1957 convention in Dallas, this study presents data on the leaching action of water on concrete pipe with special reference to the removal of lime.

●

"Ultimate Strength Design of Rectangular Concrete Members Subject to Unsymmetrical Bending," written by TUNG AU, provides charts for determining the dimensions of the equivalent compressive stress block after the dimensions of the section and the arrangement of the reinforcement are assumed.

●

An investigation to determine to what extent compressive strength and pulse velocity tests indicate the variability and compressive strength of concrete as it exists in a series of columns is reported by M. F. KAPLAN in "A Study of Compressive Strength and Ultrasonic Pulse Velocity Relationships for Concrete in Columns."

●

S. J. CHAMBERLIN presents the results of a study to determine the effect of spacing of lapped bars on bond and the effect of length of lap on the load-carrying capacity of small beams in "Spacing of Spliced Bars in Beams."

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

Secretary, Technical Activities Committee
AMERICAN CONCRETE INSTITUTE
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Tentative Recommendations for Prestressed Concrete*

Reported by ACI-ASCE Joint Committee 323

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This report was submitted to letter ballot of the committee which consists of 33 members; 32 members returned their ballots, of whom 32 have voted affirmatively.

†These members constitute executive group of committee.

‡Deceased: served as committee chairman from 1949 until 1955.

SYNOPSIS

A guide to design and construction of safe, serviceable, linear structural members prestressed with high strength steel. Emphasis is on flexural members —beams, girders, and slabs. Most of the recommendations are applicable to both buildings and bridges. Design chapter treats: loading; allowable stress; prestress loss; flexure and shear; bond and anchorage; composite construction; continuity; end blocks; fire resistance; and cover and spacing of prestressing steel. Concrete, grout, prestressing steel, anchorages, and splices are covered in the section on materials. Construction section includes: transportation, placing, and curing of concrete; forms, shoring, and falsework; placement of prestressing steel and application of the prestressing force; grouting; and handling and erection.

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CHAPTER 1—INTRODUCTION

101—OBJECTIVE

The objective of this report is to recommend those practices in design and construction which will result in prestressed concrete structures that are comparable both in safety and in serviceability to constructions in other materials now commonly used.

This report constitutes a recommended practice, not a building code or specification. Since it was not written as a code, its use or interpretation as one will not serve the best interests of either the public or the engineering profession. Recommendations contained in the report are presented solely for the guidance and information of professional engineers. Safety and economy of structures in prestressed concrete will depend as much on the intelligence and integrity of engineers preparing the design and supervising or carrying out the construction as on the degree to which these recommendations are followed.

102—SCOPE

102.1—Linear prestressing

This report is confined in scope to linear structural members involving prestressing with high strength steel; circularly prestressed members such as tanks or pipes are not covered. These types of construction have been excluded for two reasons. They have been designed and constructed in this country for a great number of years and procedures have been developed on the basis of research and experience which have proved successful in practice. Design and construction of tanks and pipes in prestressed concrete are confined to a relatively small group of specialists and are not likely to be attempted by persons outside that group. For these reasons there seems to be no immediate need for recommendations regarding circularly prestressed structures.

102.2—Flexural members

For the most part, recommendations in this report relate to flexural members—beams, girders, and slabs. Other structural forms, such as columns, ties, arches, shells, trusses, pavements, etc., are treated only briefly or not at all. In some of these cases, such as columns or ties, the principles involved in design are essentially simple and no need was felt to include them in this report. In other cases, insufficient information was available either from research or experience to permit recommendations to be made at this time. This lack of information is due in some instances to the complexity of the type of structure involved and in others to the infrequency of its use in this country.

102.3—Buildings and bridges

These recommendations are intended to apply to both buildings and bridges. The form and nature of this report are such that almost all recommendations made apply without differentiation to both types of structures. Where this is not the case, separate recommendations are made for buildings and bridges.

103—ACCEPTANCE TESTS

It is recognized by the committee that unusual types of construction, design, or materials may be used in such a manner that these recommendations are not applicable or may not have been complied with. Such structures may be adequate for the purpose intended. In these cases it is recommended that tests be made to verify design.

104—NOTATION

104.1—General

Symbols are assembled into sections pertaining to groups of associated terms. The list comprises only the symbols in this report. No attempt is made to present a complete notation for design of prestressed concrete.

104.2—Dimensions and cross-sectional constants

A_b = bearing area of anchor plate of post-tensioning steel

A_c = maximum area of the portion of the anchorage surface that is geometrically similar to and concentric with the area of the bearing plate of post-tensioning steel

A_s = area of main prestressing tensile steel

A_s' = area of conventional tensile steel

A_{sf} = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange

A_{sr} = steel area required to develop the ultimate compressive strength of the web of a flanged section

A_v = area of web reinforcement placed perpendicular to the axis of the member

b = width of flange of a flanged member or width of a rectangular member

b' = width of web of a flanged member

d = distance from extreme compressive fiber to centroid of the prestressing force

I = moment of inertia about the centroid of the cross section

j = ratio of distance between centroid of compression and centroid of tension to the depth d

p = A_s/bd; ratio of prestressing steel

p' = ratio of conventional reinforcement

pf_s'/f_c' = percentage index

s = longitudinal spacing of web reinforcement

t = average thickness of the flange of a flanged member

Q = statical moment of cross section area, above or below the level being investigated for shear, about the centroid

104.3—Loads

D = effect of dead load

L = effect of design live load including impact, where applicable

W = effect of wind load, or earthquake load, or traction forces

V_c = shear carried by concrete

104.4—Stresses and strains

E_c = flexural modulus of elasticity of concrete

E_s = modulus of elasticity of prestressing steel

f_c' = compressive strength of concrete at 28 days

f'_{ci} = compressive strength of concrete at time of initial prestress

f_{cp} = permissible compressive concrete stress on bearing area under anchor plate of post-tensioning steel

f_s' = ultimate strength of prestressing steel

f_{se} = effective steel prestress after losses

f_{si} = initial stress in prestressing steel after seating of the anchorage

f_{su} = stress in prestressing steel at ultimate load

f_{sy} = nominal yield point stress of prestressing steel

f_t' = flexural tensile strength of concrete; modulus of rupture

f_y' = yield point stress of conventional reinforcing steel

k_2 = ratio of distance between extreme compressive fiber and center of compression to depth to neutral axis

k_1k_3 = ratio of average compressive concrete stress to cylinder strength, f_c'

n = ratio of E_s/E_c

u_d = strain in concrete due to creep

u_e = strain in concrete due to elastic shortening

u_s = strain in concrete due to shrinkage

v = shearing stress

δ_1 = ratio of loss in steel stress due to relaxation of prestressing steel

δ_2 = ratio of loss in steel stress due to friction during prestressing

104.5—Friction during prestressing

e = base of Naperian logarithms
K = friction wobble coefficient per ft of prestressing steel
T_o = steel stress at jacking end
T_x = steel stress at any point x
μ = friction curvature coefficient
α = total angular change of prestressing steel profile in radians from jacking end to point x
L = length of prestressing steel element from jacking end to point x

CHAPTER 2—DESIGN

201—GENERAL CONSIDERATIONS

201.1—Purpose

The purpose of design is to define a structure that can be constructed economically, that will perform satisfactorily under service conditions, and will have an adequate ultimate load capacity.

201.2—Mode of failure

Ultimate strength should be governed preferably by elongation of the prestressing steel rather than by shear, bond, or concrete compression.

201.3—Design theory

The elastic theory should be used at design loads with internal stresses limited to recommended values. The ultimate strength theory also should be applied to insure that ultimate capacity provides the recommended load factors.

202—SPECIAL CONSIDERATIONS

202.1—Loading conditions

Consideration should be given to all critical loading conditions in design including those that occur during fabrication, handling, transportation, erection, and construction.

202.2—Deflections

Camber and deflection may be design limitations and should be investigated for both short and long time effects.

202.3—Length changes

Length changes of concrete due to prestress and other causes should be investigated for both short and long time effects.

202.4—Reversal of loading effects

Where reversal of moment or shear may occur it should be considered in the design.

202.5—Buckling

General buckling due to prestressing can occur only over the length between points of contact of the prestressing steel with the concrete.

General buckling of an entire member or local buckling of thin webs and flanges under external loads may occur in prestressed concrete as in members made of other materials and should be provided for in design.

203—ASSUMPTIONS

203.1—Basic assumptions

The following assumptions may be made for design purposes:

a. Strains vary linearly over the depth of the member throughout the entire load range.
b. Before cracking, stress is linearly proportional to strain.
c. After cracking, tension in the concrete is neglected.

203.2—Modulus of elasticity

When accurate values for modulus of elasticity are not available, the following values may be used as a guide:

a. Flexural modulus of elasticity of concrete E_c, in psi, may be assumed to be 1,800,000 plus 500 times the cylinder strength at the age considered. Actual values may vary as much as 25 percent from those given by the foregoing expression. This expression is not applicable to lightweight concrete, for which E_c should be determined by test.

b. Modulus of elasticity of steel, in psi, may be assumed to be 29,000,000 for cold drawn wire, 27,000,000 for 7-wire strand, 25,000,000 for strand with more than 7 wires, and 27,000,000 for alloy steel bars.

203.3—Deflections

Deflection or camber under short time loading may be computed using values of E_c obtained as described in Section 203.2.a.

Deflection associated with dead load, prestress, and live loads sustained for a long time may be computed on the assumption that the corresponding concrete strains are increased as a result of creep. The increase in strain may vary from 100 percent of the elastic strain in very humid atmosphere to 300 percent of the elastic strain in very dry atmosphere. These values may not pertain to concrete made with lightweight aggregates.

204—LOADING STAGES

204.1—Loading

Loading stages listed in the following sections should be investigated. No attempt is made to list all significant loading stages that may occur. Stages listed are those that normally affect the design.

204.2—Initial prestress

Prestressing forces are applied in prearranged sequence and sometimes in stages. If prestressing forces are not counteracted by the effect of the dead load of the member, or if the stressing operation is accompanied by temporary eccentricities, concrete stresses should be investigated.

204.3—Initial prestress plus dead load of member

For determination of concrete stresses at this stage, losses in prestress are those which occur during and immediately after transfer of prestress.

204.4—Transportation and erection

Support conditions for precast members during transportation and erection may differ from those during service loads. Handling stresses should be included together with prestress and dead load. Losses in initial prestress up to time of handling should be considered.

204.5—Design load

This stage includes stress due to effective prestress after losses, dead loads, and maximum specified live load.

204.6—Cracking load

Complete freedom from cracking may or may not be necessary at any particular loading stage. Type and function of the structure and type, frequency, and magnitude of live loads should be considered.

204.7—Temporary overload

This stage refers to any large live load in excess of design load, which is of short-time duration and expected to occur infrequently during life of structure. For such a load, stresses may exceed those recommended for design load but elastic recovery must be assured.

204.8—Ultimate load

Ultimate load is that load which applied statically in a single application causes failure. Such a large load would never intentionally be placed on the structure, but it is used as a measure of safety. In statically determinate structures, failure will occur at a single cross section. In statically indeterminate structures, the load which causes moment in one section to reach its ultimate value may not be sufficient to cause failure of the structure because of moment redistribution. Since it is not always possible to predict that full redistribution will take place in accordance with limit design, it is suggested for the time being that moments be determined by elastic analysis.

205—LOAD FACTORS

205.1—General

A load factor is a multiple of the design loads used to insure safety of the structure.

205.2—Cracking load factors

If cracking of concrete is undesirable, load factors for cracking load should be chosen to reflect the greatest load that can be expected during life of structure.

Formation of a crack under temporary overload may not be objectionable. If reopening such a crack under subsequent design load is objectionable it may be avoided by proper choice of concrete stress permitted for cracking load.

205.3—Ultimate load factors

The ultimate load capacity should be computed since stresses are not linearly proportional to external forces and moments throughout the entire load range. For the present, it is recommended that moments, shears, and thrusts produced by external loads and prestressing forces be investigated by elastic analysis.

The load factors recommended are believed to be consistent with current viewpoints. It may be desirable to modify or expand the load factor formulas to fit special conditions that may occur in unusual structures, extremely long spans, or for unique loadings. Deviations from the recommended values should be substantiated by suitable investigations.

205.3.1—*Buildings*

For the present, to correlate prestressed concrete with reinforced concrete practice in current use, the committee recommends that ultimate load capacity be investigated to insure meeting the following requirements:

$$1.2\,D + 2.4\,L$$
or $$1.8\,(D + L)$$
or $$1.2\,D + 2.4\,L + 0.6\,W$$
or $$1.2\,D + 0.6\,L + 2.4\,W$$
or $$1.8\,(D + L + \tfrac{1}{2}\,W)$$
or $$1.8\,(D + \tfrac{1}{2}\,L + W)$$

whichever is greater.

205.3.2—*Highway bridges*

The following load factors for highway bridges are recommended by a subgroup appointed by American Association of State Highway Officials, Committee on Bridges and Structures.*

$$1.5\,D + 2.5\,L$$

The committee is not prepared at this time to make recommendations for load factors involving the effect of lateral loads on bridges.

205.3.3—*Railway bridges*

Ultimate load factors for railway bridges are currently being studied by the American Railway Engineering Association. The committee is not prepared to recommend such factors at this time.

206—REPETITIVE LOADS

206.1—General

Ultimate strength of concrete or steel subjected to repetitive loading may be less than static strength because of the phenomenon of fatigue. Full importance of fatigue in prestressed concrete members has not yet been determined. Fatigue failure may occur in concrete, steel, anchorages, splices, or bond.

*These load factors are considered adequate for spans of moderate length, simply supported. For exceptionally long spans and for continuous members, special investigation to consider a possible increase in load factors is recommended.

206.2—Concrete

Fatigue strength of concrete in both tension and compression depends on magnitude of stress, range of stress variation, and number of loading cycles. Since high stresses and stress ranges are common, fatigue should be considered when repetition of loading cycles may occur.

Fatigue failure is unlikely if the allowable stresses of Section 207.3.2 are not exceeded and there is no reversal of stress. If a large number of overloads are anticipated a reduction in the safety factor may occur.

206.3—Prestressing steel

Fatigue strength of prestressing steel depends on magnitude and range of stress, and number of cycles of loading. Minimum stress is the effective prestress. Maximum stress and range of stress depend on magnitude of live loads or overloads that may be repeated. Range of stress under service loads will usually be small unless concrete is cracked. Cracking may occur if tension is permitted in concrete. Fatigue failure of steel should be considered in such cases, especially when a high percentage of ultimate strength is used for prestress.

Devices for splicing steel may contain strain concentrations that lower fatigue strength. Consideration should be given to fatigue whenever splices are used.

206.4—Anchorages

If steel is fully bonded, no difficulty should be expected in the anchorage or end bearing as the result of repetitive loads. With unbonded steel, fluctuations in stress due to repeated service loads or overloads are transmitted directly to anchorages and fatigue strength of the anchorage will require special consideration.

206.5—Bond

Failure of bond under repetitive loading is unlikely unless the member is cracked under design loads or a significant number of repetitions of overload. High bond stresses adjacent to cracks may be a source of progressive failure under repeated loads.

206.6—Shear and diagonal tension

Since inclined cracks may form under repetitive loading at appreciably smaller stresses than under static loading, web reinforcement should always be provided in members subjected to repetitive loading.

206.7—Design recommendations

Fatigue should not result in a reduction of strength if the following recommendations are observed. When the recommendations cannot be followed, fatigue strength of all elements comprising the prestressed member should be considered.

a. Flexural compressive concrete stress should not exceed $0.4f_c'$ under either design load or an overload that may be repeated many times.

b. Tension should not be permitted in concrete at the critical cross section under either design load or overloads that may be repeated a large number of times.

c. Reversal of stress should not occur under repeated loads.

d. Prestressing steel should be bonded.

e. Web reinforcement should be provided.

207—ALLOWABLE STEEL AND CONCRETE STRESSES

207.1—Prestressing steel

207.1.1—*Temporary stresses*

Under normal design loads stress in prestressing steel will almost always be less than stress at initial prestress. Stress at the anchorage immediately after seating has been effected should not exceed $0.70f_s'$ for material having stress-strain properties defined in Chapter 3. Overstressing for a short period of time to $0.80f_s'$ may be permitted provided the stress, after seating of anchorage occurs, does not exceed $0.70f_s'$.

207.1.2—*Stress at design loads*

Effective steel stress after losses described in Section 208 should not exceed:

$$0.60f_s' \text{ or } 0.80f_{sy}$$

whichever is smaller.

207.2—Non-prestressed reinforcement

Non-prestressed reinforcement provided to resist tension in conformance with requirements of Section 207.3.1.b.2 may be assumed stressed to 20,000 psi.

207.3—Concrete

207.3.1—*Temporary stresses*

Concrete stress in psi before losses due to creep and shrinkage should not exceed the following:

a. Compression

For pretensioned members	$0.60f'_{ci}$
For post-tensioned members	$0.55f'_{ci}$

b. Tension

1. For members without non-prestressed reinforcement:

Single element	$3\sqrt{f'_{ci}}$
Segmental element	zero

2. For members with non-prestressed reinforcement provided to resist the tensile force in the concrete, computed on the basis of an uncracked section:

Single element	$6\sqrt{f'_{ci}}$
Segmental element	$3\sqrt{f'_{ci}}$

207.3.2—*Stresses at design loads*

After full prestress losses, stresses in psi should not exceed the following:

a. Compression
   1. Single element
      a. Bridge members ..... $0.40f_c'$
      b. Building members ..... $0.45f_c'$
   2. Segmental elements
      a. Bridge members ..... $0.40f_c'$
      b. Building members ..... $0.45f_c'$

b. Flexural tension in the precompressed tensile zone
   1. Single element
      a. Bridge members ..... zero
      b. Pretensioned building elements not exposed to weather or corrosive atmosphere ..... $6\sqrt{f_c'}$
      c. Post-tensioned bonded elements not exposed to weather or corrosive atmosphere ..... $3\sqrt{f_c'}$
   2. Segmental elements
      a. Bridge members ..... zero
      b. Building members ..... zero

Allowable flexural tension of $6\sqrt{f_c'}$ in Section 207.3.2.b.1.b may be exceeded provided it is shown by tests that the structure will behave properly under service conditions and meet any necessary requirement for cracking load or temporary overload.

207.3.3—*Stress at cracking load*

Flexural tensile strength (modulus of rupture) should preferably be determined by test. When test data are not available the ultimate flexural tensile stress in psi may be assumed as:

$$f_t' = 7.5\sqrt{f_c'}$$

For lightweight concrete, f_t' should be determined by tests.

207.3.4—*Anchorage bearing stresses*

The maximum allowable stress at post-tensioning anchorage in end blocks adequately reinforced in conformance with Section 214.4 may be assumed as:

$$f_{cp} = 0.6 f'_{ci} \sqrt[3]{A_c/A_b}$$

where A_b = bearing area of the anchor plate.
A_c = maximum area of portion of the member that is geometrically similar to and concentric with the area of bearing plate.

The allowable value of f_{cp} should not exceed f'_{ci}.

208—LOSS OF PRESTRESS

208.1—Introduction

Initial prestress is that stress in steel which exists immediately after seating of anchorage. Stress diminishes with time and finally reaches a stable condition of effective prestress assumed to be permanent.

208.2—Sources of prestress loss

208.2.1—*Friction loss in post-tensioned steel*

If post-tensioned steel is draped, or irregularities exist in alignment of ducts, steel stress will be less within the member than at the jack because of friction between prestressing steel and duct. Magnitude of this friction should be estimated for design and verified during stressing operation.

Friction loss may be estimated from an analysis of forces exerted by prestressing steel on duct. One method for determination of friction loss at any point is given below.

$$T_o = T_x \, e^{(KL + \mu\alpha)}$$

where T_o = steel stress at jacking end

T_x = steel stress at point x

e = base of Naperian logarithms

K = friction wobble coefficient per ft of prestressing steel

L = length of prestressing steel element from jacking end to point x in ft

μ = friction curvature coefficient

α = total angular change of prestressing steel element in radians from jack to point x

For small values of KL and $\mu\alpha$ the following formula may be used:

$$T_o = T_x (1 + KL + \mu\alpha)$$

The following values of K and μ are typical and may be used as a guide. They may vary appreciably with duct material and method of construction. Values of K and μ used in design should be indicated on the plans for guidance in selection of materials and methods that will produce results approaching the assumed values.

Type of steel	Type of duct or sheath	Usual range of observed values		Suggested design values	
		K	μ	K	μ
Wire cables	Bright metal sheathing	0.0005-0.0030	0.15-0.35	0.0020	0.30
	Galvanized metal sheathing			0.0015	0.25
	Greased or asphalt-coated and wrapped	0.0030	0.25-0.35	0.0020	0.30
High strength bars	Bright metal sheathing	0.0001-0.0005	0.08-0.30	0.0003	0.20
	Galvanized metal sheathing			0.0002	0.15
Galvanized strand	Bright metal sheathing	0.0005-0.0020	0.15-0.30	0.0015	0.25
	Galvanized metal sheathing			0.0010	0.20

Workmanship in placing, supporting, tying, and fabricating prestressing elements and ducts influences the magnitude of wobble factor, K. The larger the sheath or duct in relation to the size of prestressing steel element, the smaller K will be. With normal placing tolerances wobble effect may be neglected if sheath is 1 in. greater in diameter than prestressing steel element.

Effect of overestimating friction loss should be considered since excessive prestress may cause undesirable permanent stress conditions. Underestimating friction loss may result in an error in computing cracking load and deflection.

208.2.2—*Elastic shortening of concrete*

Loss of prestress caused by elastic shortening of the concrete occurs in prestressed concrete members. This loss equals $n(\Delta f_c)$. For pretensioned concrete, Δf_c is the concrete stress at the center of gravity of the prestressing steel for which the losses are being computed. For post-tensioned concrete where the steel elements may not be tensioned simultaneously, Δf_c is the average concrete stress along one prestressing element from end to end of the beam caused by subsequent post-tensioning of adjacent elements.

208.2.3—*Shrinkage of concrete*

Shrinkage depends on many variables. Unit shrinkage strain may vary from near 0 to 0.0005. A value between 0.0002 and 0.0003 is commonly used for calculation of prestress loss. Shrinkage loss may be greater in pretensioned members where the prestress is transferred to the concrete at an earlier age than is usual for post-tensioned members. Shrinkage of lightweight concrete may be greater than the values obtained with the above factors.

208.2.4—*Creep of concrete*

Creep is the time-dependent strain of concrete caused by stress. For pretensioned and post-tensioned bonded members, concrete stress is taken at center of gravity of prestressing steel under effect of prestress and permanent loads (normal conditions of unloaded structure).

In post-tensioned unbonded members, stress is the average concrete stress along the profile of center of gravity of prestressing steel under the effect of prestress and permanent loads. Additional strain due to creep may be assumed to vary from 100 percent of elastic strain for concrete in very humid atmosphere to 300 percent of elastic strain in very dry atmosphere.

Creep of some lightweight concretes may be greater than indicated above.

208.2.5—*Relaxation of steel stress*

Loss of stress due to relaxation of prestressing steel should be provided for in design in accordance with test data furnished by the steel manufacturer. Loss due to relaxation depends primarily on properties of the steel and initial prestress. This loss is generally assumed in the range of 2 to 8 percent of initial steel stress.

208.3—Alternate procedures for estimating prestress losses

Two methods are suggested for estimating prestress losses. Method 1 should be used when individual losses may be predicted with reasonable accuracy. Method 2 applies when specific loss data are lacking.

The ultimate strength is not significantly affected by the magnitude of steel stress loss. An error in choosing the loss is reflected in the cracking load and amount of camber.

208.3.1—*Method 1*

The total stress loss in prestressing steel:

$$\Delta f_s = (u_s + u_e + u_d)\, E_s + \delta_1 f_{si} + \delta_2 f_{si}$$

208.3.2—*Method 2*

Loss in steel stress not including friction loss may be assumed as follows:

Pretensioning	35,000 psi
Post-tensioning	25,000 psi

For camber calculations these values may be excessive.

208.4—Lightweight concrete

Losses due to concrete shrinkage, elastic shortening, and creep should be based on results of tests made with the lightweight aggregate to be used.

209—FLEXURE

209.1—Stresses due to dead, live, and impact loads

Prestressed concrete members may be assumed to function as uncracked members subjected to combined axial and bending forces provided stresses do not exceed those given in Section 207.

In calculations of section properties prior to grouting, areas of the open ducts should be deducted unless relatively small. The transformed area of bonded reinforcement may be included in pretensioned members and post-tensioned members after grouting.

For calculation of stress due to prestress in T-beams no definite recommendations are made at this time, but attention should be given to the possibility that the entire available flange width may be included in calculation of section properties.

209.2—Ultimate flexural strength

209.2.1—*General method*

(a) *Rectangular sections*—For rectangular sections or flanged sections in which the neutral axis lies within the flange, ultimate flexural strength may be expressed as:

$$M_u = A_s f_{su}\, d \left(1 - \frac{k_2}{k_1 k_3} \frac{p f_{su}}{f_c'}\right) \qquad \text{(a)}$$

where f_{su} = average stress in prestressing reinforcement at ultimate load
d = depth to centroid of force
k_2 = ratio of distance between extreme compressive fiber and center of compression to the depth to neutral axis
k_1k_3 = ratio of average compressive concrete stress to the cylinder strength, f_c'

The results of numerous tests have shown that the factor k_2/k_1k_3 may be taken equal to 0.6 for members and materials considered in this report. Determination of the value of f_{su} requires knowledge of the stress-strain characteristics of the prestressing steel, effective prestress and crushing strain of the concrete. Assumptions must be made regarding the relation between steel and concrete strains. These assumptions will be different for bonded and unbonded construction.

The ultimate moment may be computed from Eq. (a) whenever sufficient information is available for the determination of f_{su}. The approximate method of Section 209.2.2 may be used if the required conditions are satisfied.

(b) *Flanged sections*—If a flange thickness is less than $1.4dpf_{su}/f_c'$, the neutral axis will usually fall outside the flange and the following approximate expression for ultimate moment should be used:

$$M_u = A_{sr}f_{su}\,d\left(1 - 0.6\frac{A_{sr}f_{su}}{b'df_c'}\right) + 0.85f_c'\,(b - b')t\,(d - 0.5t) \quad \text{.....(b)}$$

where $A_{sr} = A_s - A_{sf}$ = the steel area required to develop the ultimate compressive strength of the web of a flanged section

$A_{sf} = 0.85f_c'\,(b - b')t/f_{su}$ = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

t = average thickness of flange

The expressions for f_{su} given in Section 209.2.2 may be used if the required conditions are satisfied.

209.2.2—*Approximate method*

The following approximate expressions for f_{su} may be used in Eq. (a) and (b) of Section 209.2.1 provided the following conditions are satisfied:

1. The stress-strain properties of the prestressing steel are reasonably similar to those described in Section 304.
2. The effective prestress after losses is not less than $0.5f_s'$.

(a) *Bonded members*

$$f_{su} = f_s'\left(1 - 0.5\frac{pf_s'}{f_c'}\right)$$

(b) *Unbonded members*—Ultimate flexural strength in unbonded members generally occurs at lower values of steel stress than in bonded members. Wide variations between stress levels reported by different investigators reflect the fact that several factors influence the stress developed by unbonded steel at ultimate moment. These factors include: magnitude of effective prestress, profile of the prestressing steel, shape of the bending moment diagram, length/depth ratio of the member, magnitude of the friction coefficient between the prestressing steel and duct, and amount of bonded non-prestressed supplementary steel.

Unless the proper value of f_{su} is known from tests of members closely approximating proposed construction with respect to the several factors listed in the preceding paragraph, it is recommended that:

$$f_{su} = f_{se} + 15{,}000$$

209.2.3—*Maximum steel percentage*

To avoid approaching the condition of over-reinforced beams for which the ultimate flexural strength becomes dependent on the concrete strength, the ratio of prestressing steel preferably should be such that pf_{su}/f_c' for rectangular sections, and $A_{sr}f_{su}/b'df_c'$ for flanged sections are not more than 0.30.

If a steel ratio in excess of this amount is used, the ultimate flexural moment shall be taken as not greater than the following values when either the general or approximate method of calculation is used.

(a) *Rectangular sections*

$$M_u = 0.25\, f_c'bd^2$$

(b) *Flanged sections*—If the flange thickness is less than 1.4 dpf_{su}/f_c' the neutral axis will usually fall outside the flange and the following formula is recommended.

$$M_u = 0.25bd^2f_c' + 0.85f_c'\,(b - b')t(d - 0.5t)$$

209.2.4—*Non-prestressed reinforcement in conjunction with prestressing steel*

209.2.4.1—*Conventional reinforcement*—Non-prestressed conventional reinforcement may be considered to contribute to the tensile force in the beam at ultimate moment an amount equal to its area times its yield point provided that

$$\frac{pf_{su}}{f_c'} + \frac{p'f_y'}{f_c'} \text{ does not exceed } 0.3$$

where f_y' = yield point of conventional reinforcement
p' = ratio of conventional reinforcement

209.2.4.2—*High tensile strength reinforcement*—If untensioned prestressing steel or other high tensile strength reinforcement is used in conjunction with prestressed reinforcement, the ultimate moment should be calculated by means of the general method of Section 209.2.1.

210—SHEAR

210.1—General

210.1.1—*Ultimate strength*

It is essential that shear failure should not occur before ultimate flexural strength required in Section 209.2 is developed. If this condition is satisfied, it is unnecessary to investigate shear or principal tensile stresses at design loads.

210.1.2—*Inclined cracking*

Formation of inclined cracks precedes failure in shear. They are caused by inclined principal tensile stresses that are the resultant of shearing stresses and normal bending stresses. Compressive prestress reduces the principal tensile stress thereby increasing the load necessary to cause inclined cracks. The use of thin webs will increase inclined stresses.

210.1.3—*Conditions for shear failure*

The resistance to formation of inclined cracks is greater with larger prestress and increasing web thickness. The significance of inclined cracks is less with low ultimate flexural strength caused by low ratio of reinforcement. Their significance is also less with low shear/moment ratios. If inclined cracks occur in an unreinforced web, sudden failure by shear is almost certain. If the web is adequately reinforced, ultimate flexural strength can be developed.

210.2—Web reinforcement

210.2.1—*Critical percentage of tensile steel*

Experimental data, although limited, indicate that inclined tension cracks will not form and web reinforcement will not be required if the following condition is satisfied:

$$\frac{pf_s'}{f_c'} \geq 0.3\frac{f_{se}}{f_s'}\frac{b'}{b}$$

where b' = thickness of web; b = width of flange corresponding to that used in computing p

This expression may be conservative for members having span/depth ratios greater than about 15 or for uniformly loaded members. In such cases, web reinforcement may not be required even though the percentage index, pf_s'/f_c', exceeds that given in the above expression. The omission of web reinforcement in such members may be allowed when justified by tests.

210.2.2—*Design of web reinforcement*

The amount of web reinforcement necessary to develop required ultimate flexural capacity is a function of the difference between inclined cracking load and ultimate load in flexure. This difference varies rather widely as a function of prestress force, web thickness, amount of tensile reinforcement, and shear/moment ratio but is usually smaller for prestressed concrete than for conventional reinforced concrete. Current design procedures for web reinforcement in reinforced concrete are conservative for prestressed concrete. Available test data indicate that the following expression for area of web reinforcement, with its factor of ½, will give reasonably conservative results

for prestressed members of usual dimensions and properties. Since the formula does not involve the prestress force it may not be conservative for very low prestress or where only a portion of the reinforcement is stressed. For such cases it may be necessary to increase the factor of ½ as the member approaches the condition of conventionally reinforced concrete.

$$A_v = \tfrac{1}{2}\frac{(V_u - V_c)s}{f_v' jd}$$

where A_v = area of web reinforcement at spacing s, placed perpendicular to the axis of the member
V_u = shear due to specified ultimate load and effect of prestressing
V_c = $0.06 f_c' b' jd$ but not more than $180 b' jd$
s = longitudinal spacing of web reinforcement
f_v' = yield strength of web reinforcement

210.2.3—*Minimum quantity of web reinforcement*

Because of the nature and limited knowledge of shear failures, it is suggested that some web reinforcement be provided even though the criterion of Section 210.2.1 is satisfied.

Where the web reinforcement is designed by Section 210.2.2, the minimum amount of web reinforcement should be $A_v = 0.0025 b's$. This requirement may be excessive for members with unusually thick webs and the amount of web reinforcement may be reduced if tests demonstrate that the member can develop its required flexural capacity.

Heavily loaded members with thin webs and relatively small span/depth ratios, such as highway bridge girders and crane girders should have web reinforcement (see Section 206.6).

210.2.4—*Spacing of web reinforcement*

The spacing of web reinforcement should not exceed three-quarters the depth of the member. In members with relatively thin webs, spacing should preferably not exceed the clear height of the web.

210.2.5—*Critical sections for shear*

Because formation of inclined cracks reduces flexural capacity the critical sections for shear will usually not be near the ends of the span where the shear is a maximum but at some point away from the ends in a region of high moment.

For the design of web reinforcement in simply supported members carrying moving loads, it is recommended that shear be investigated only within the middle half of the span length. The web reinforcement required at the quarter-points should then be used throughout the outer quarters of the span.

For simply supported members carrying only uniformly distributed load, the maximum web reinforcement may be taken as that required at a distance from the support equal to the depth of member. This amount of web reinforcement should be provided from this point to the end of member. In the middle third of the span length, the amount of web reinforcement provided should not be less than that required at third-points of the span.

211—BOND AND ANCHORAGE

211.1—Pretensioning

211.1.1—*Prestress transfer bond*

Bond between the pretensioned steel and concrete is necessary to establish a prestress in the concrete. The transfer of force from the steel to the concrete takes place in a finite length in the end region of a member and the function of the resulting bond, termed "prestress transfer bond," is anchorage of prestressing steel. Prestressing force varies from near zero at the end to a maximum value some distance from the end.

Transfer length will generally be of minor significance in long members, but it should be considered for short members or those in which the loading conditions may cause cracking in or near the region of prestress transfer.

211.1.2—*Flexural bond*

Flexural bond is the bond stress developed as a consequence of flexure. Bond stress at design loads in uncracked members is usually not critical since the increase in steel stress resulting from flexure is usually not significant. If cracking is anticipated under design loads, bond stress should be given special consideration.

211.1.3—*Significance of bond stress at ultimate load*

Bond failure should not occur prior to the development of the required ultimate flexural capacity.

For span lengths usually associated with prestressed concrete, bond failure is not a significant design factor. Bond adequacy in extremely short members should be investigated by test.

The factors affecting bond are concrete strength, perimeter shape, area and surface condition of prestressing steel, stress in the steel at ultimate strength, length of transfer zone, and superimposed load pattern.

212—COMPOSITE CONSTRUCTION

212.1—Introduction

Prestressed concrete structures of composite construction are comprised of prestressed concrete elements and plain or conventionally reinforced concrete elements interconnected in such a manner that the two components function as an integral unit. The prestressed elements may be pretensioned or post-tensioned and may be precast or cast in place. The plain or reinforced concrete elements are usually cast in place.

212.2—Interaction

212.2.1—*Shear connection*

To insure integral action of a composite structure at all loads, a connection should be provided between the component elements of the structure capable of performing two functions:

(1) To transfer shear without slip along the contact surfaces, and
(2) To prevent separation of the elements in a direction perpendicular to the contact surfaces.

212.2.2—*Transfer of shear*

Slip may be prevented and shear transferred along the contact surfaces either by bond or by shear keys. It should be assumed that the entire shear is transferred either by bond or by shear keys.

212.2.3—*Anchorage against separation*

Mechanical anchorage in the form of vertical ties should be provided to prevent separation of the component elements in the direction perpendicular to the contact surfaces. Web reinforcement or steel dowels adequately embedded on each side of the contact surface will provide satisfactory mechanical anchorage.

212.3—Design of shear connection

212.3.1—*Loading stage*

The shear connection should be designed for ultimate load.

212.3.2—*Magnitude and transfer of ultimate shear*

The shear at any point along the contact surface may be computed by the usual method as $v = (V_u Q)/I$. If the bond capacity is less than the computed shear, full width shear keys should be provided throughout the length of the member. Keys should be proportioned according to concrete strength of each component of the composite member.*

212.3.3—*Capacity of bond*

The following values are suggested for ultimate bond resistance of the contact surfaces.

When minimum steel tie requirements of Section 212.3.4 are followed ..... 75 psi
When minimum steel tie requirements of Section 212.3.4 are followed and the contact surface on the precast element is artificially roughened ..... 150 psi
When additional steel ties in excess of the requirements of Section 212.3.4 are used and the contact surface of the precast element is artificially roughened ..... 225 psi

212.3.4—*Vertical ties*

In the absence of experimental information on the capacity of vertical ties it is recommended that all web reinforcement be extended into the cast-in-place concrete.

Spacing of vertical ties should not exceed four times the minimum thickness of the composite elements, or 24 in. whichever is less. The total area of vertical ties should not be less than that provided by two #3 bars spaced at 12 in.

*Lack of experimental data makes the committee hesitate to recommend a shear stress at the root of a key. Indications are that for keys on bridge girders in current use shear stress at the root of a key as high as $0.3f_c'$ would sometimes be required to transmit ultimate shear force.

For light pretensioned members such as those used for building floors not subjected to repetitive loads the above minimum requirements may be too severe. The committee is not prepared to recommend an amount or spacing of steel for this type member.

212.4—Design of composite structures

212.4.1—*Design of composite section.*

Physical properties of the composite section should be computed on the assumption of complete interaction between component elements. For structures composed of concretes of different qualities, the area of one of the component elements should be transformed in accordance with the ratio of the two moduli of elasticity.

212.4.2—*Beam and slab construction*

If the structure is composed of beams with a cast-in-place slab placed on top of the beams, effective slab width should be computed in the same manner as for integral T-beams.

212.4.3—*Allowable stress with different concrete strengths*

In structures composed of elements with different concrete strengths, the allowable stresses should be governed by strength of the portion under consideration.

212.4.4—*Superposition of stress*

Stresses may be superposed in design calculations that involve elastic stresses. Superposition of stresses should not be used in computing ultimate strength since inelastic action of the material is involved.

212.4.5—*Stress after structure becomes integral*

The properties of the composite cross section should be used in computing stresses due to loads applied after the structure becomes integral.

212.4.6—*Shrinkage stresses*

In structures with a cast-in-place slab supported by precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of precast beams. Stresses due to differential shrinkage are important only insofar as they affect cracking load. When cracking load is significant, such stresses should be added to the effects of loads.

212.4.7—*Ultimate strength*

Ultimate strength of a composite section should be computed in the same manner as ultimate strength of an integral member of the same shape.

213—CONTINUITY

213.1—Determination of moments, shears, and thrusts

Moments, shears, and thrusts produced by external loads and prestressing force should be determined by elastic analysis. Effects of axial deformation should be considered. Determination of effects produced by the prestressing

forces should take into account the restraint of attached structural elements and supports.

213.2—Stresses

Allowable stresses are those recommended in Section 207.

213.2.1—*Prestress*

When prestressing is to be applied in more than one stage, the internal stresses should be investigated at each stage.

213.3—Frictional losses

Frictional losses in continuous post-tensioned steel may be more significant than in simply supported members.

213.4—Ultimate strength

The ultimate strength of a continuous member should be evaluated not only at points of maximum moment, but also at intermediate points. In applying ultimate load factors where dead load causes effects opposite to those of live load, consideration should be given to load factor combinations in which dead load factor may equal one. It is recommended that moment redistribution not be considered in design at the present time.

214—END BLOCKS

214.1—Purpose

An enlarged end section, called an end block, may be required to transmit concentrated prestressing forces in a shaped member from the anchorage area to the basic cross section.

End blocks may be required to provide sufficient area for bearing of anchorages in post-tensioned design. They may be needed to transmit vertical and lateral forces to supports and to facilitate end detailing.

214.2—Requirements

In pretensioned members with large concentrated eccentric prestressing elements, end blocks should be used. For lightly pretensioned members, or members of approximately rectangular shape, end blocks may be omitted. However, reinforcement should always be provided in the anchorage zone.

In post-tensioned, shaped members, end blocks should be provided.

214.3—Proportioning

End blocks are usually proportioned by experience. Depending on the degree of concentration and eccentricity of the prestressing force at the end surface, the length of the end block should be from one-half the depth of the member to the full depth. In general, shallow members should have an end block length equal to the depth, and deep beams should have an end block length equal to three-quarters of the depth. Length of an end block can be considered as the distance from beginning of anchorage area to the point where the end block intersects the narrowest width of member.

214.4—Reinforcement

Reinforcing is necessary to resist tensile bursting and spalling forces induced by the concentrated loads of the prestressing steel. A reinforcing grid with both vertical and horizontal steel in the plane of the cross section should be provided directly beneath anchorages to resist spalling forces. Closely spaced reinforcement should be placed both vertically and horizontally throughout the length of the end block to resist tensile forces.

215—FIRE RESISTANCE

215.1—General

The fire resistance of both prestressed concrete and reinforced concrete is subject to the same general limitations. One is the rate of heat transmission through the concrete from the surface exposed to fire to the unexposed surface. The other is the reduction of steel strength at the temperatures induced in the steel during the test. Either limitation may govern.

215.2—Heat transmission

Since the rate of heat transmission through prestressed concrete is similar to that of reinforced concrete of the same composition, the critical dimensions to control temperature rise at the unexposed surface will be the same in prestressed or reinforced concrete members.

215.3—Load-carrying capacity

The ability of the structure to carry required loads during fire test depends largely on thickness of cover over prestressing steel. The following minimum thicknesses of concrete cover on prestressing steel and end anchorages are recommended for various fire ratings:

Hour rating	1 hr	2 hr	3 hr	4 hr
Minimum concrete cover	1½ in.	2½ in.	3 in.	4 in.

Data now available are insufficient to make recommendations for such factors as shape of cross section, type and arrangement of prestressing steel. The cover thicknesses recommended are believed to be conservative.

216—COVER AND SPACING OF PRESTRESSING STEEL

216.1—Cover

The following minimum clear concrete covers are recommended for prestressing steel, ducts, and non-prestressed steel.

	Minimum concrete cover
Concrete surfaces exposed to weather	1½ in.
Concrete surfaces in contact with ground	2 in.
Beams and girders not exposed to weather	
Prestressing steel, and main reinforcing steel	1½ in.
Stirrups and ties	1 in.
Slabs and joists not exposed to weather	¾ in.

216.2—Spacing at ends

216.2.1—*Spacing of pretensioning steel*

Minimum horizontal or vertical clear spacing between pretensioning steel elements at ends of members should be three times the diameter of the steel or 1⅓ times the maximum size of coarse aggregate, whichever is greater.

216.2.2—*Spacing of post-tensioning ducts*

The clear space between conduits at the ends should be a minimum of 1½ in. or 1½ times the maximum size of coarse aggregate, whichever is greater.

216.2.3—*Dimensions of post-tensioning ducts*

When steel is placed inside conduits which are to be filled with cement grout, such conduits should have a minimum inside diameter ¼ in. larger than the diameter of the prestressing steel.

216.3—Draped prestressing steel

When prestressing steel is placed in a curved or deflected position, steel or conduits may be bundled together in the middle third of the span length provided the minimum spacing recommended in Section 216.2.1 and 216.2.2 is maintained for a minimum distance of 3 ft at each end of member. The committee is not prepared to suggest limits for the number of conduits or prestressing steel elements that may be bundled horizontally and vertically. Excessive bundling may lead to insufficient bond capacity in pretensioned members, resulting in bond slip.

CHAPTER 3—MATERIALS

301—INTRODUCTION

The nature and economics of prestressed concrete construction require the use of high strength materials. Ability to sustain high stresses with a minimum of time-dependent change in stress or strain is essential.

These requirements are more severe than those for conventionally reinforced concrete. Highest standards of manufacture and construction should be observed. Prior to adoption of new materials, sufficient test data should be obtained to verify properties assumed in design.

302—CONCRETE

302.1—Scope

Particular attention should be given to properties of individual materials used in prestressed concrete and their effect on compressive strength, modulus of elasticity, drying shrinkage, creep, bond strength, and uniformity of concrete in place.

When new materials and methods are employed, trial mix investigations should include tests for drying shrinkage, creep, and modulus of elasticity.

302.2—Materials

302.2.1—*Portland cement*

Portland cement should conform to one of the following:

Specifications for Portland Cement (ASTM C 150)
Specifications for Air-Entraining Portland Cement (ASTM C 175)
Specifications for Portland Blast Furnace Slag Cement (ASTM C 205)
Specifications for Portland-Pozzolan Cement (ASTM C 340)

302.2.2—*Concrete aggregates*

Concrete aggregates should conform to one of the following:

Specifications for Concrete Aggregates (ASTM C 33)
Specifications for Lightweight Aggregates for Structural Concrete (ASTM C 330)

Mineral composition and soundness of aggregates may have a marked influence on compressive strength, modulus of elasticity, drying shrinkage, and creep.

Concretes made with some lightweight aggregates may exhibit a lower modulus of elasticity, greater creep and drying shrinkage than do concretes of the same strength made with aggregates of normal weight.

The range of properties possible in the same concrete mix with different lightweight aggregates may be large. Therefore, it is recommended that test data should be obtained for compressive strength, modulus of elasticity, drying shrinkage, creep, modulus of rupture, and bond.

302.2.3—*Water*

Water for mixing concrete should be clean and free of injurious quantities of substances harmful to concrete or to prestressing steel. Sea water should not be used for making prestressed concrete.

302.2.4—*Admixtures*

Certain admixtures may be beneficial to fresh or hardened concrete. However, admixtures should not be used until shown by test to have no harmful effect on the steel or concrete.

The use of calcium chloride or an admixture containing calcium chloride is not recommended where it may come in contact with prestressing steel.

302.3—Proportioning, batching and mixing

The proportioning of materials, batching, and mixing of concrete for prestressing should be done in accordance with the *ACI Manual of Concrete Inspection*, the U. S. Bureau of Reclamation *Concrete Manual*, or other comparable regulations including ACI Standards "Recommended Practice for Winter Concreting (ACI 604-56)," "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42);" and "Standard Specifications for Ready-Mix Concrete" (ASTM C 94).

Available materials should be proportioned to produce concrete meeting specification requirements with a minimum water content. Slump of fresh

concrete should be as low as feasible. Cement, sand, and narrow-size ranges of coarse aggregate should be separately batched by weight. Water and some liquid admixtures may be batched by volume with accurate measuring equipment. Close control of all materials and operations is essential.

302.4—Strength

The strength required at given ages should be specified by the designer. Controlled concrete should be used and tested in accordance with Section 304 as modified by Section A602(f) of "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

303—GROUT

303.1—General

When required by job specifications, post-tensioned steel should be grouted to completely fill the void surrounding the prestressing steel with a portland cement grout to insure high flexural bond strength and provide permanent protection for the steel.

303.2—Materials

Grout should be made of either (a) cement and water or (b) cement, fine sand, and water. Mix (a) should be used where the cavity is very small. Either Mix (a) or Mix (b) may be used where the cavity is relatively large. Admixtures should conform to recommendations of Section 303.2.4.

303.2.1—*Portland cement*

Same as Section 302.2.1.

303.2.2—*Sand*

Sand should preferably be a natural quartz sand meeting "Tentative Specification for Aggregate for Masonry Mortar (ASTM C 144)," except for gradation requirements. The sand should pass a No. 30 sieve, about 50 percent should pass a No. 50 sieve, and about 20 percent should pass a No. 100 sieve.

303.2.3—*Water*

Same as Section 302.2.3.

303.2.4—*Admixtures*

Certain admixtures may be beneficial to fresh or hardened grout. However, no admixture should be used until shown by test to have no harmful effect on the steel or grout.

Calcium chloride or an admixture containing calcium chloride is not recommended for use in grouting post-tensioned members.

303.3—Proportioning

Proportions of grouting materials should be based on results of tests made on fresh and hardened grout prior to beginning work. Grout should have the consistency of thick cream or heavy paint. When permitted to stand until setting takes place, grout should neither bleed nor segregate.

304—PRESTRESSING STEEL

304.1—General

High tensile strength steel is required in prestressed concrete to provide necessary internal concrete stresses after losses have occurred. The following four types are in common use:

(a) High tensile strength single wire, applied in the form of assemblies made up of two or more substantially parallel wires. They may be used for either pretensioning or post-tensioning purposes.

(b) Small diameter, high strength strand, shop fabricated, is usually made up of six wires spiraled around a center wire. Small diameter strand is normally, though not exclusively, used for pretensioning purposes.

(c) Large diameter high strength strand is usually shop fabricated with factory attached end fittings for post-tensioned construction. It has 7, 19, 37, or more individual wires.

(d) High strength alloy steel bars are produced by a cold stretching or drawing process. They are currently available in diameters ranging from ½ to 1⅛ in. Alloy steel bars are used principally for post-tensioned construction.

Each type of prestressing steel should be made to distinctly separate specifications, of which the following sections give a general description.*

304.2—High tensile strength single wire

High tensile strength single wire is generally made from high carbon steel hot rolled into rods. It is then heat treated by a process termed "patenting" and cold drawn to produce the required final tensile strength. In its most commonly used form the wire is then stress relieved by a controlled time-temperature treatment that improves elastic properties within the tensile range usually employed in prestressing concrete. It also produces a straighter, more easily handled wire.

High tensile strength wire produced by the oil tempering process is not recommended for use in prestressed concrete.

304.2.1—*Ultimate tensile strength*

High tensile strength wire for prestressed concrete is made to minimum tensile strengths as high as 250,000 psi for a diameter of 0.196 in. Higher tensile strengths are available at smaller diameters and lower tensile strengths at larger diameters.

304.2.2—*Shape of stress-strain curve*

Stress relieved wire for prestressing should display a high yield strength and a reasonable elongation before rupture. Minimum yield strength at 1 percent elongation under test load should be equal to 85 percent of specified ultimate tensile strength. Minimum elongation after rupture should be 4 percent in 10 in. Elongation tests should conform to "Specification for Mechanical Testing of Steel Products" (ASTM A 370-54T).

*The American Society for Testing Materials is currently formulating specifications for prestressing steels.

304.2.3—*Ductility*

Wire for prestressing should be capable of a reasonable amount of cold deformation without failure. It should have a minimum reduction in cross-sectional area of 30 percent at rupture.

304.2.4—*Creep and relaxation*

Data concerning typical creep and stress relaxation properties of the material should be obtained from the manufacturer. Special acceptance tests for individual lots are usually expensive and unnecessary.

Creep tests and short-term relaxation tests do not necessarily represent long-time stress relaxation characteristics.

304.3—Small diameter high strength wire strand

Small diameter high strength strand is normally made of seven wires. A straight center wire is enclosed tightly by six spirally wound outer wires. Because of its small diameter, strand can be given a final stress-relieving treatment similar to that for single wires. This treatment improves elasticity and handling characteristics. Acceptance tests, when required, should be made on the strand rather than single wires.

Physical properties should be based on the total metallic area of all the individual wires. Ultimate tensile strength, shape of stress-strain curve, ductility, creep and relaxation should be the same as described in Section 304.2 (high tensile strength single wire) except as follows:

(a) Minimum elongation at rupture, 3.5 percent in 24 in.

(b) Minimum yield strength at 1 percent elongation under test load equal to 85 percent of specified ultimate tensile strength.

304.4—Large diameter high strength wire strand

Large strand may be made of 7, 19, 37, or more galvanized or uncoated hard-drawn wires, spirally wound. Galvanized strand is most commonly used.

Because large diameter strand cannot be given a final stress-relieving treatment, some of its physical properties differ from those of wire or small strand. Acceptance tests, when required, should be based on properties of the strand rather than individual wires.

304.5—Cold stretched high strength alloy steel bars

These bars are usually made from alloy steel designated AISI 5160 or AISI 9260. After hot rolling, the bars are either heat treated or cold worked. Each bar is then cold stretched to a minimum of 90 percent of the specified ultimate strength.

304.5.1—*Ultimate tensile strength*

High strength alloy steel bars are produced with a minimum tensile strength of 145,000 psi for all diameters.

304.5.2—*Shape of stress-strain curve*

High strength bars for prestressing should have a minimum yield strength at 0.2 percent permanent strain equal to 90 percent of the specified ultimate

tensile strength. Minimum elongation after rupture should be 4 percent in a length of 20 diameters.

304.5.3—*Ductility*

Bars for prestressing should be capable of a reasonable amount of cold deformation without failure. The bar should have a reduction of area of not less than 15 percent at rupture.

304.5.4—*Creep and relaxation*

Data concerning typical creep and stress relaxation properties of the material should be obtained from the manufacturer. Special acceptance tests for individual lots are usually expensive and unnecessary.

Creep tests and short-term relaxation tests do not necessarily represent long-time stress relaxation characteristics.

304.6—Corrosion

Since prestressing steels are susceptible to corrosion, they should be protected during storage, transit, and construction.

The term stress corrosion is applied to the embrittlement of steel that occurs under the combined effects of high stress and some corrosive environments. It may take place without apparent surface impairment.

Normally, steel cast in concrete or properly grouted will not be subject to such corrosion. When post-tensioned steel is not grouted, special precautions should be taken to protect the steel (see Section 404.3.2).

305—ANCHORAGES AND SPLICES

305.1—General

Anchorages for post-tensioning elements now in general use consist of:

Threaded ends and wedge anchors for bars; factory attached end fittings for large diameter strand; button-head, sandwich plate, and conical wedges for parallel lay wire systems; and conical wedges for small diameter strand.

Splices are used primarily for bars and consist of threaded couplings.

305.2—Ultimate strength

Anchorages and splices should be capable of developing the ultimate strength of attached steel elements without excessive deformation.

305.3—Anchorage set

Movement of prestressing steel in anchorage during seating should be stated by the manufacturer and substantiated by test data.

CHAPTER 4—CONSTRUCTION

401—INTRODUCTION

This chapter outlines construction procedures that should result in sound and durable structures.

Prestressed concrete members are composed of high strength concrete and steel. Design stresses are closely controlled, but behavior in service depends

upon the specified concrete being properly placed in forms of the correct dimensions around accurately positioned prestressing steel or ductwork for steel. Construction requires accuracy and care. Deviation from careful workmanship may result in an unsafe structure and should not be condoned.

402—TRANSPORTING, PLACING, AND CURING OF CONCRETE

402.1—General

Quality of the finished concrete members depends on care used in transporting, placing, and curing. Recommended practice is outlined in "Building Code Requirements for Reinforced Concrete (ACI 318-56)," Sections 403-406, and "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)."

402.2—Placing

Low slump, high cement content mixes should be placed in the shortest possible time after mixing is completed to prevent loss of workability.

Concrete should be deposited close to its final position. The method of placement should be such that segregation will not occur.

402.3—Vibration

Internal or external vibration or both are usually necessary to produce dense, well-compacted concrete.

Vibrators should not be used to move concrete horizontally in the form. Overvibration should also be avoided.

When internal vibration is used, vibrator heads should be smaller than the minimum distance between ducts or prestressing steel. Care must be exercised to avoid damage to or misalignment of ducts for post-tensioning steel.

Vibration is not a substitute for workability. Judgment should be used in specifying slump, and approved methods of vibration used to achieve maximum compaction.

402.4—Construction joints

In long cast-in-place members the use of construction joints is recommended (1) to reduce cracking near columns caused by settlement or movement of shoring and falsework, and (2) to allow for shrinkage. In general, joints should be placed near falsework supports.

Construction joints preferably should be perpendicular to prestressing steel. Joints should not be made parallel to prestressing steel unless the provisions of Section 212 (composite construction) are followed.

402.5—Curing

Curing should start soon after finishing. If high temperature curing is used, an initial setting time prior to application of heat should be required. Curing should continue until the required strength for application of the prestress force is reached. Fresh concrete should be protected from rain or the rapid

loss of moisture prior to the curing period. Rapid drying should be prevented until the final design strength is obtained.

When high temperature curing is used, the rate of heating and cooling should be controlled to reduce thermal shock to the concrete.

Where identical precast members are required, curing conditions should be uniform to maintain proper quality control.

402.6—Protection from freezing

During periods of freezing temperatures, ungrouted ducts should be blown clear of water or protected against freezing.

403—FORMS, SHORING, AND FALSEWORK

403.1—General

Quality of concrete members depends on the care used in constructing forms and falsework. Correct practices outlined in "Building Code Requirements for Reinforced Concrete (ACI 318-56)" Sections 501 and 502 are recommended.

403.2—Special requirements

Forms for pretensioned members should be constructed to permit movement of the member without damage during release of the prestressing force.

Forms for post-tensioned members should be constructed to minimize restraint to elastic shortening during prestressing and shrinkage. Deflection of members due to the prestressing force and deformation of falsework should be considered in design. Form supports may be removed when sufficient prestressing has been applied to carry dead load, formwork carried by the member, and anticipated construction loads.

404—PLACEMENT OF PRESTRESSING STEEL AND APPLICATION OF PRESTRESSING FORCE

404.1—General

The location of the center of gravity of the prestressing steel, initial and final prestressing force, and the assumed losses due to creep, shrinkage, elastic shortening, and friction shown on the plans are based on the use of specified materials. Other materials not specified but capable of producing the same results may be used with approval of the engineer.

Unless tolerances for location of the prestressing steel are shown, a variation of $\pm$ $\frac{1}{8}$ in. to $\pm$ $\frac{1}{4}$ in. depending on size of the member, is suggested as maximum permissible.

404.2—Pretensioning steel

404.2.1—*General*

Steel should be kept clean and dry. Foreign matter, grease, oil, paint, and loose rust should be removed prior to casting concrete. A light coat of

rust is permissible and sometimes preferable provided loose rust has been removed and the surface of the steel is not pitted.

404.2.2—*Measurement of prestressing force*

Pretensioning force should be determined by measuring elongation and checking jack pressure on a calibrated gage. Measurement of elongation will usually give more consistent results. When there is a difference of over 5 percent between the steel stress determined from elongation and from the gage reading, the cause of the discrepancy should be ascertained and corrected.

If several wires or strands are stretched simultaneously, provision must be made to induce the same initial stress in each.

404.2.3—*Transfer of prestressing force*

The force in the prestressing steel should be transferred to the concrete smoothly and gradually. If the force in the wires or strands is transferred individually, a sequence of release should be established by the engineer to avoid subjecting the member to unanticipated stresses. Any variation in this sequence should be submitted to the engineer for approval.

404.2.4—*Protection*

Ends of pretensioning steel exposed to weather or corrosive atmosphere should be protected by a coating of asphaltic material. They should preferably be recessed in the member, coated with asphaltic material and covered with mortar.

404.3—Post-tensioning steel

404.3.1—*General*

The steel should be kept clean and dry. For bonded construction, foreign matter, grease, oil, paint, and loose rust should be removed prior to placing steel in ducts. A light coat of rust is permissible provided loose rust has been removed and the surface of the steel is not pitted.

404.3.2—*Protection*

For general use in unbonded construction, galvanizing may be considered to protect the steel from corrosion when coated with grease or asphalt-impregnated material and enclosed in a sheath. Uncoated galvanized steel may be used when it is accessible for inspection and points of bearing are equipped with special shoes to prevent damage to the galvanizing.

If wrappings and coatings are used on nongalvanized steel, the coating should protect the steel from corrosion during shipment, storage, construction, and after the steel is in place. It should permit movement of steel during stressing with minimum friction. The method of protection should be specified or approved by the engineer.

Anchorage and end fittings should be given protective treatment consistent with that given the prestressing steel. They should preferably be recessed in the member and covered with mortar.

404.3.3—*Placement of steel and enclosures*

Ducts or enclosures for prestressing steel are formed in the concrete using tubing, metallic casings, or other materials. They should be positioned and secured to maintain the prestressing steel within the allowable placement tolerances.

For bonded construction, ducts or duct-forming devices should be free from grease, paint, or other foreign matter. Ducts should be protected against entrance of foreign matter prior to grouting.

Anchorage hardware to be cast in the member should be firmly fastened to forms in the proper location.

404.3.4—*Measurement of the prestressing force*

Values of total elongation, corrected for assumed friction loss and anchorage set, and corresponding jack pressures at various increments of prestress should be supplied by the engineer. When a difference of over 5 percent exists between steel stress determined from the corrected elongation and from corresponding gage reading, stressing operation should cease. If the cause of the discrepancy is neither faulty measurement nor equipment, the engineer should be consulted.

404.3.4.1—*Factors influencing friction*—As prestressing force is applied, friction between prestressing steel and curved enclosure reduces steel stress at points away from the jack. The amount of friction loss is a function of degree of curvature, type and length of prestressing steel, duct material, presence of friction reducing agents, accuracy of placing the duct, and degree of disturbance during concrete placement.

It is the responsibility of the contractor to be aware of these factors. He should use materials specified and insure that the quality of workmanship results in accurate duct positioning with minimum displacement during construction.

404.3.5—*Prestressing in stages*

When the prestressing force is to be applied in more than one stage, excessive concrete stresses should be avoided during intermediate stages. The engineer will designate location and magnitude of the forces to be used for each stage and allowable external loads that may be placed on the member. The contractor should be aware of the significance of overloading the member.

404.3.6—*Anchorage set*

For friction type anchorages the manufacturer or supplier should state the amount of slip normally expected in seating the anchorage device.

404.3.7—*Effect of temperature*

Changes in temperature should have little effect on prestressing reinforcement unless there is a significant temperature differential between concrete and steel.

405—GROUTING

405.1—General

When grouting is specified for post-tensioned members it should completely fill all enclosure voids.

405.2—Mixing

Grout should be mixed in a mechanical mixer. Immediately after mixing, it should be passed through a strainer into pumping equipment which provides for recirculation. Grout should be pumped into the duct as soon as possible after mixing but may be pumped as long as it retains the proper consistency.

405.3—Arrangement of grout pipes

Ducts must be provided with entrance and discharge ports, each of which can be closed. Extension pipes may be used when necessary.

For long members, grout may be introduced at one end until it discharges from an intermediate point. The point of application may then be moved successively forward. Grout may be introduced at an intermediate point if discharge ports are provided at duct ends. The sequence of grouting should be planned to insure complete filling. Devices for bleeding air may be required at high points of the duct profile.

405.4—Test for passage of grout

Free passage of grout from entrance to discharge port must be assured. Tests may be made by pumping water, air, or other fluids through the duct.

405.5—Application of grout

Grout should be applied continuously until it flows steadily from the discharge port indicating removal of trapped air and water. The discharge port should then be closed and grouting pressure maintained for the length of time necessary to insure complete filling of the void. The entrance should then be closed and the pumping nozzle removed.

405.6—Protection against freezing

Adequate precautions must be taken to prevent freezing fresh grout.

406—HANDLING AND ERECTION

Where precast members are specified, methods of handling and/or the sequence of erection should be indicated. When these are not indicated on the plans, the contractor should submit for approval the location of pick-up points, minimum concrete strength when handled, method of transporting, and sequence of erection.

Discussion of this report should reach ACI headquarters in triplicate by Apr. 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

Multistory Lift-Slab Construction*

By W. SEFTON†

SYNOPSIS

Outlines the general procedure for lift-slab work, citing main advantages and disadvantages. Describes in some detail the slabs, collars, jacks, fittings, jack rods, columns, and anchorage and foundation problems. Principles governing lifting sequence are noted; an example shows lifting sequence for 12-story building. Cost factors are treated, and current Canadian trends in lift-slab applications are given.

INTRODUCTION

Lift-slab construction has developed because of the annoyance of having to build a timber structure first, and then casting a concrete structure in it. The cost of building two structures to get a single end product is reflected both in the extra materials involved and the extra time and labor required to erect and subsequently demolish and remove the timber structure. Lift slab is by now an established method and requires little introductory explanation.

Construction procedure

The floors are designed as flat plates without column heads or drop panels. In other words, neither the top nor the bottom surface of the slab has any projections. The floors of one lift-slab structure in Calgary are 8 in. thick, and the steel box columns are 9-in. to 8-in. square, made up of two 8 x 8 x 1-in. to ½-in. angles welded toe-to-toe along their length.

The columns are erected first on prepared foundations. There are no steel beams or concrete beams, but to minimize the unsupported length of the columns temporary angle bracing is provided at the top of columns only.

Around each column a specially made steel collar is provided, one for each floor to be cast (Fig. 1). These collars are placed around the columns before erection and each one is incorporated with the concrete of its floor.

The ground floor is cast directly on the basement floor which has been machine troweled to a smooth surface. The surface of the basement-floor concrete is first brushed with a liquid wax and then dusted with a foundry dusting powder. The reinforcement is placed on the floor and all conduit work is placed at this stage. Concrete for the floor is then cast and machine troweled. This concrete in turn is brushed with a liquid wax and the dusting powder is applied. The next floor is then cast on the ground floor. There is

*Adapted from a report presented at the ACI regional meeting, Montreal, Quebec, Oct. 24, 1956. Title No. 54-31 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 7, Jan. 1958, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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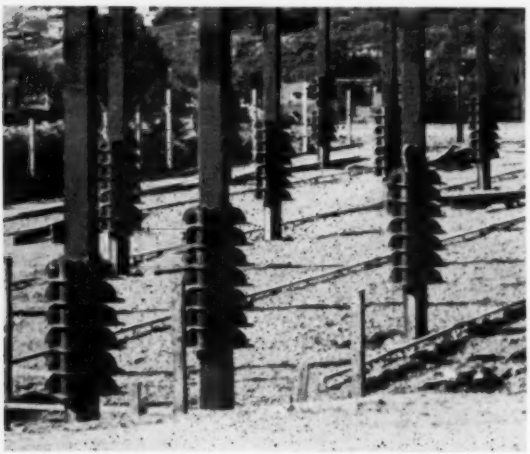

Fig. 1—Collars in place on columns before concreting begins

no physical division with material such as building paper. Concrete is cast directly on hardened concrete. This sequence is continued until all floors and the roof are cast (Fig. 2). The cycle between finishing one floor and finishing the floor next above need only be 2 days. No extra time is necessary for curing. The top slab, usually the roof slab, is allowed to reach the required strength. This takes 3 to 10 days and then the jacks are set up over each column. These are hydraulically operated and lift through special screwed rods passing through the jack and into the collars at each column. Jacking commences and the slabs are lifted to their required positions (Fig. 3).

Lift slab advantages

First and foremost, speed of erection is considerable. Two slabs can be lifted in a day with final fixing to follow. Each slab can be cast in 2 to 4 days. Formwork is almost entirely eliminated. The lower surface of each slab is as good as the machine troweled polished surface of the floor on which it was cast. Plastering of ceiling is thus entirely eliminated and a plastic paint is the only finish applied to the ceilings.

All heating and electrical work can be incorporated in the slabs on the ground (Fig. 4). All concreting is done on the ground. Beams can be entirely eliminated, giving the architect almost complete freedom in his internal arrangements. Floors can be cast in the winter at temperatures of −30 F and the heating is confined to the area at ground level where the slabs are made. Finally economy—cost of certain lift-slab work at Calgary saved more than 30 percent of the cost currently prevailing in Canada for the structural portion of the work.

Disadvantages

This is a new system of construction; just as steel and reinforced concrete required radical revisions in the thinking of architects and owners, so does

the lift-slab system of construction. To use the system to its fullest advantage, architects and engineers have to learn those features which give the maximum of economy in the completed building.

For example, at the present stage of development, it is desirable to have columns set in from the edges of slabs about 2 or 3 ft minimum, and in fact this can be taken advantage of by increasing the cantilevers substantially to provide unobstructed window space.

Since the h/t for columns receiving any bending is limited by the ACI Building Code to 20, concrete columns higher than about 30 ft involve design and construction problems, and structural steel is usually used. Requirements of the AISC Code for unbraced cantilevered columns limit the height of a lift for economically practical sizes of steel columns to about 45 ft at a time, and so slabs are lifted and parked (Fig. 5) and then lifted and parked again, as often as required by the over-all height of the building. Certain types and sizes of columns, such as 12-in. inside diameter pipes are impossible to use without the latest equipment.

Lifting is conveniently limited to 12 to 18 jacks at a time since this is the limiting number of jacks handled by the usual jacking control unit. This is an advantage as well as a disadvantage, in that on a job with 36 columns or less, lifts are done on half the structure at a time, and this enables an efficient and speedy sequence to be adopted.

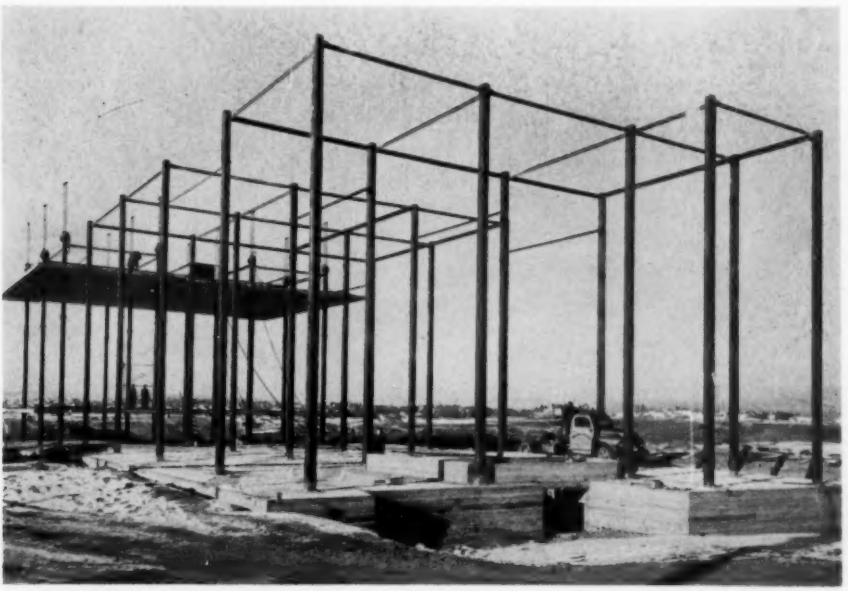

Fig. 2—All slabs stacked in place as cast. Lifting has begun on slabs in background

BASIC LIFT-SLAB COMPONENTS

Slabs

The reinforced concrete slabs must be such that one can be cast over another, the top of one being under the bottom of the next. This has led to use of the flat plate in most cases since it is the simplest to make, and the top of one slab is the only form required to construct the next one except for a small amount of perimeter formwork.

Many other slabs are used too, including the waffle slab and ribbed slab. Cardboard box forms have increased this trend recently. There are also two almost identical school auditoriums in Ontario where 70-ft upturned post-tensioned beams have been lifted, carrying with them a thin reinforced concrete slab flush with the bottoms of the beams.

The slab is usually designed either by the use of the flat slab empirical moment coefficients, or by elastic analysis. The ACI Code permits the moments obtained by elastic analysis to be reduced in line with the total moment obtained by the use of the empirical coefficients. This is a progressive step. Many codes also permit a reduction of moments at columns and the transfer of this reduction to the positive moment between columns.

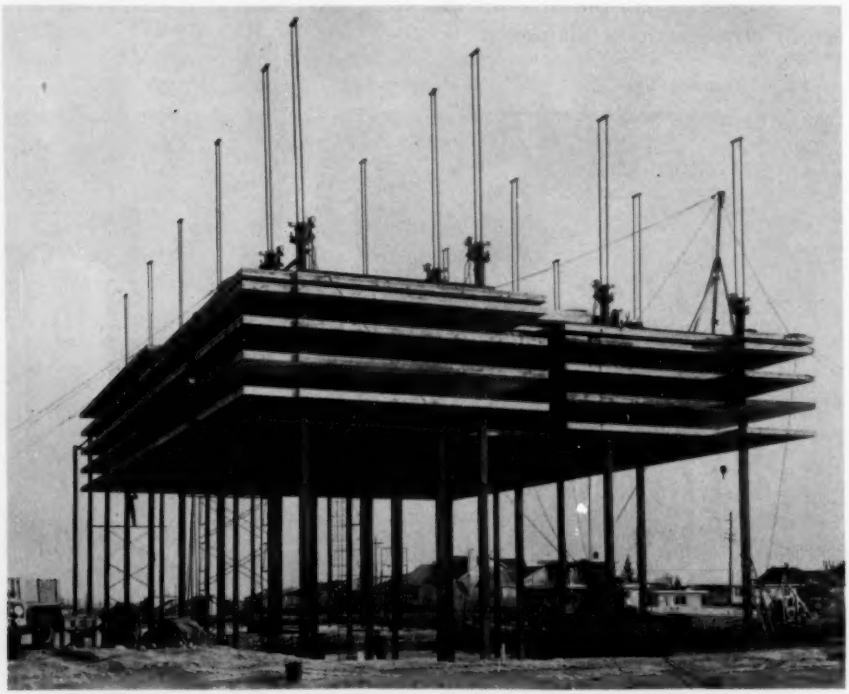

Fig. 3—Lifting operation. Upper floors stacked, third floor in final location

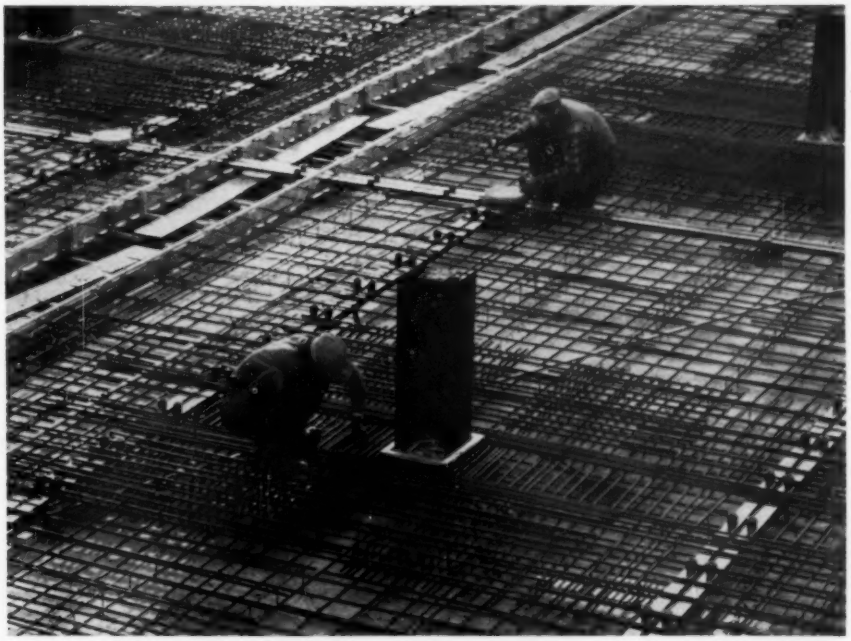

Fig. 4—Placing second-floor steel and service conduits around stub columns of a Toronto building. Note lifting collar in place around each column stub

One restrictive code provision applied to lift slab, is that which requires 25 percent of top steel to pass over the column capital. In flat plates where there is no column capital—only a collar—it is almost physically impossible to satisfy this requirement in one direction, using columns smaller than 12-in. WF's and some 10-in. WF's. A recent paper* in the ACI JOURNAL indicates possible reduction in this requirement on tests using up to 50 percent top steel passing over the column area.

Collars

Many collars now in general use in lift-slab projects are made of structural steel and this type originated in Canada. Formerly, cast steel collars were used, and some are still in use. For smaller columns the structural steel collar is still far cheaper than the cast steel collar despite the high price of structural steel today. In any case, structural steel collars are better technically to connect to structural steel columns than are cast steel collars.

The collars used on the projects described herein are in effect angles forming hollow boxes which slide vertically up the columns with 3/16-in. clearance on each side. The horizontal legs of the angles extend outward from the column

*Elstner, Richard C., and Hognestad, Eivind, "Shearing Strength of Reinforced Concrete Slabs," ACI JOURNAL, July 1956, *Proc.* V. 53, pp. 29-58.

and are stiffened by steel plate gussets. Two of the outstanding legs are drilled with holes under which the bushings of the lifting rods engage.

The basic feature of these collars is the unchangeable center-to-center distance of the lifting rod holes at 15 in. The bushings are 3⅞-in. diameter which means that inside-to-inside dimension of bushings is 11⅛ in. which makes it impossible to use box or pipe columns with a greater dimension than 11 in. across. Also a WF column with flanges and web bigger than 11 in. requires collars in which the lifting holes are *inside* the vertical legs of the

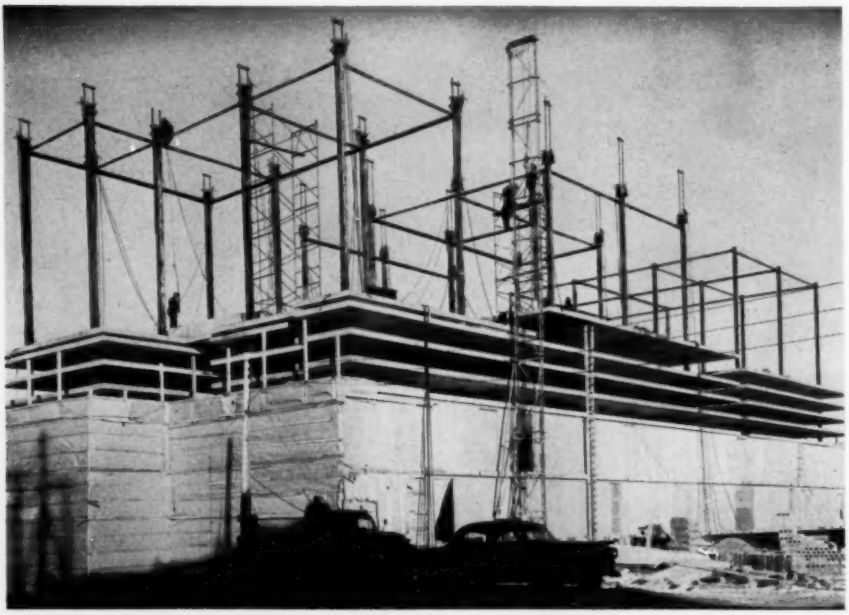

Fig. 5—Lifting parked slabs up the second lift of columns. Enclosure at base of building is for winter protection

angles and in this case the vertical legs must be at least 15 + 3⅞ in. apart, which makes the collar large and more expensive. However these internal-hole collars make it quite simple to allow 25 percent of top slab steel to pass over the column. Newer modifications in design of the collars have overcome some of these limitations.

Jacks, jack rods, and fittings

Jack rods and fittings and the jacks are not discussed in detail because they only indirectly affect the structural design. The location of the jack rods at 15-in. centers has already been discussed. The fact that the jacks are seated on top of the columns is discussed with the columns.

Fig. 6—Welding final columns to stubs

Columns

The columns (e.g., Fig. 6) in steel, which for multistory work are the common type so far, are smaller in over-all size than equivalent concrete columns and consequently the least radius of gyration is the vital factor.

The sequence of lifting is such that the critical condition for the columns in buildings of up to about eight floors, occurs when the roof slab is at the top of the columns and the next slab or group of slabs is being lifted clear of the stack on the ground. At this time the weight of two or more slabs is being taken by a column unsupported except at its ends and the load is being applied at the top. The Euler formula is based on the column length squared, and we have usually a 45-ft length of column which is 1/16 as strong as an 11-ft column of the same section, and in this case it is carrying the load of three slabs. The proper Euler coefficient applicable to this design condition is somewhat controversial.

For this reason the box or pipe column is more useful than the WF column, weight for weight, since the r_{xx} is equal to the r_{yy}. Thus an 8 x 8-in. box column made up of two 8 x 8 x ½-in. angles weighs 53 lb per ft, and was adequate for the 33-ft lift on a three-story office building recently completed in Toronto, whereas the smallest equivalent WF section had to be a 12-in.

WF weighing 65 lb per ft, and even so this has not as high a least r as the 8-in. square column. The collars for the 8-in. box are approximately half as expensive as the collars for the 12-in. WF.

Foundations

For any structure—lift-slab or otherwise—the columns must have an adequate foundation and must be properly anchored. In the case of all the lift-slab jobs for which the writer has been responsible, four $\frac{7}{8}$-in. diameter anchor bolts 1 ft 6 in. long with a 3-in. square bend have been more than adequate for each column. The writer normally ignores bending moment in the bases since the dead weight of the structure eliminates almost completely any requirement for consideration of tensile stresses due to bending.

SEQUENCE OF LIFTING

Even on two 12-story buildings designed by the writer it has been a comparatively simple matter to decide on the lifting sequence, which is the most fundamental and important single item in a lift-slab project. On smaller projects it has been no more and no less simple. The sequence for the first of these buildings is shown in Fig. 7; this building was the first lift-slab structure to receive a permit in Toronto.

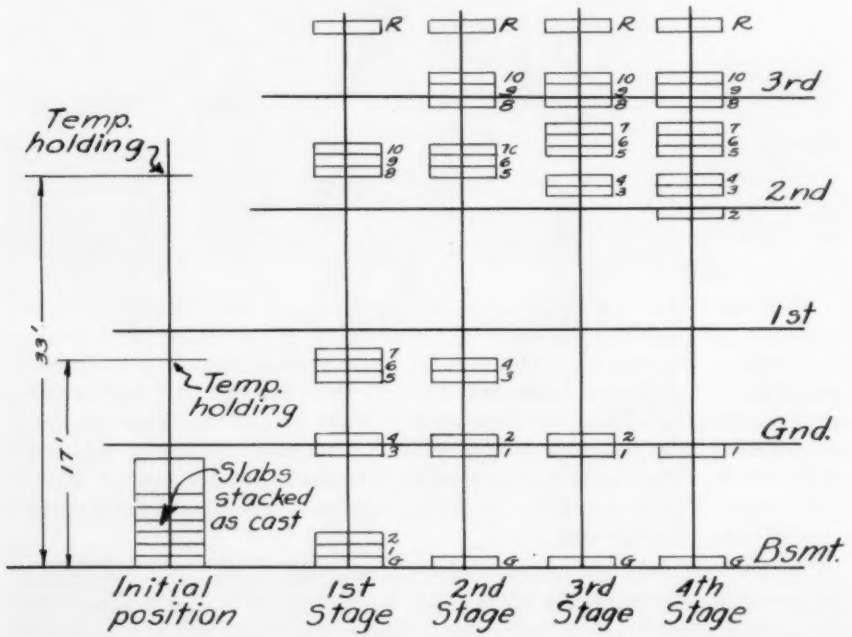

Fig. 7—Diagram of lifting sequence for 12-story building designed by author for Toronto apartments

The facts to consider are:

1. The point at which the columns may be conveniently reduced in size.
2. The height at which the Euler load is best dealt with.
3. The height required to fix finally as many of the lower slabs as possible and to stack temporarily the upper slabs conveniently for the next lift.
4. The length of jack rods available.

In fact, the usual method of approach is to limit the first column lift to something below 45 ft and to make succeeding lifts shorter and shorter as the column size reduces. Various sequences of lifting for single slabs and group of slabs are studied and the final result is chosen by trial and error. It is important to have the lifting company approve the sequence before issuing the drawings, as this company knows best the capacity and length of rods in its possession. It can also advise which sequence is likely to be cheaper.

The lifting equipment, which is constantly being improved, is designed to insure that lifting is carried out uniformly by all jacks controlled from a single control console. The specifications usually limit the maximum difference in amounts lifted between the columns in a group to ¼ in. In the older equipment

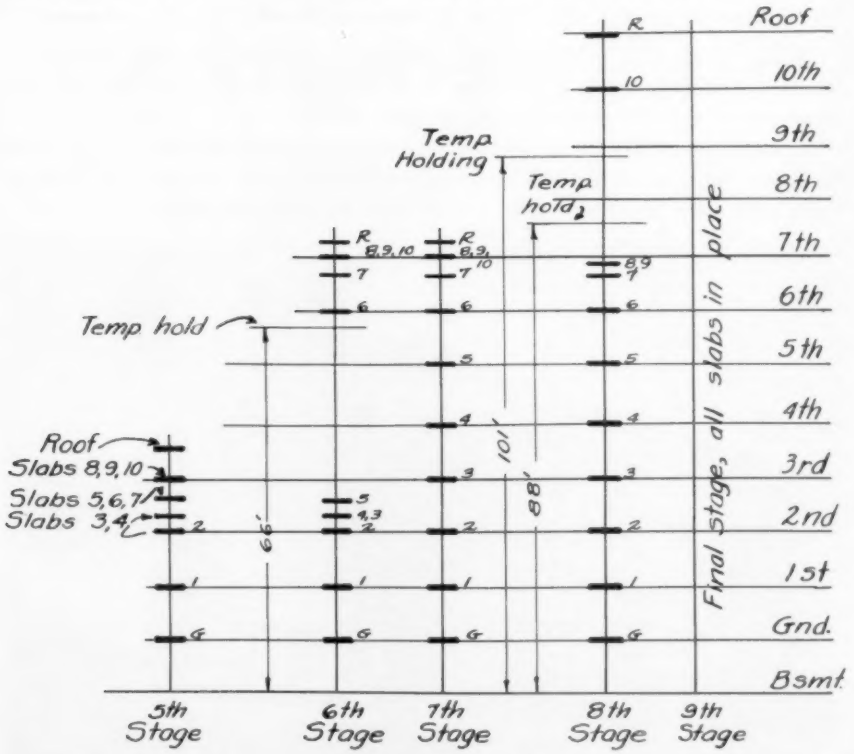

Fig. 7 (Continued)—Diagram of lifting sequence for 12-story building designed by author for Toronto apartments

still in use in many places, each set of hydraulic lines leading to each jack terminates at the console in a revolution counter type of indicator which gives a measure of the amount of lift. The operator standing at the console can control the rate of lift for each jack by a series of hand operated valves. Newer equipment does not require such a control system since the method of transmitting the hydraulic material to the jacks assures uniform lifting.

Before any slab is lifted clear of the stack on which it has been cast, the corner jacks are operated alone without the interior jacks, so that each corner is "peeled;" that is, the corner is raised from $\frac{1}{8}$ to $\frac{1}{4}$ in. to allow air to break the suction bond between the slab and the stack.

ECONOMY

The cost savings in lift-slab construction result from three main factors:

1. The elimination of slab forms saves about $0.50 per sq ft, while the lifting charge is usually about $0.30 per sq ft—a net saving of at least $0.20 per sq ft.
2. The concreting of slabs entirely at grade instead of by a hoist and in the air saves possibly between $0.10 and $0.30 per sq ft.
3. Erection speed saves an amount dependent on the value of actual time saved.

Winter protection costs are also reduced or avoided, and the writer's experience indicates that total savings of up to $1.00 per sq ft in the structure alone have resulted from the use of this method.

In addition to structural savings, the elimination of plaster ceilings saves $0.30 per sq ft, while reduced floor-to-floor heights saves volume in a building, which for most structures costs a minimum of $1.00 per cu ft.

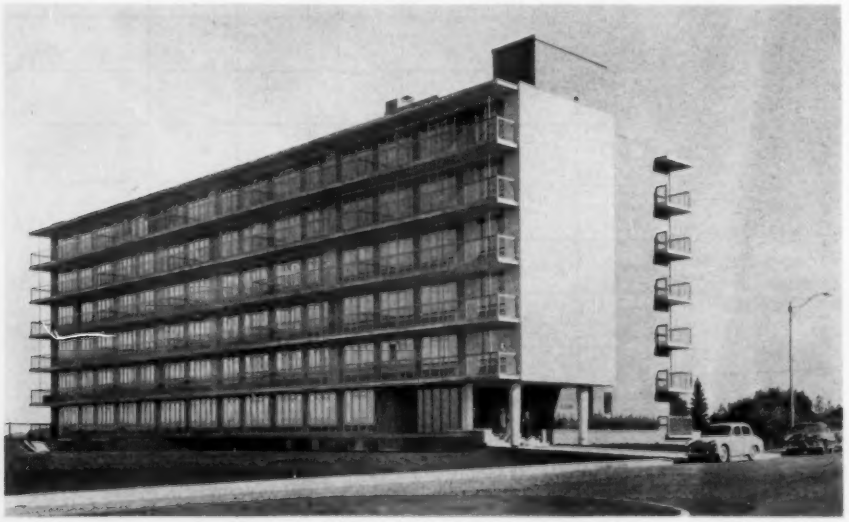

Fig. 8—Completed lift-slab building in Calgary shows use of cantilevered slabs for balconies and sunshades in residential construction

CURRENT TRENDS IN CANADA

The current trend is to more and more lift-slab buildings of all types (Fig. 8), domestic, commercial, and industrial, and this is due entirely to *cost* savings. The present status in Canada is that Winnipeg, the home of the Canadian Lift Slab Co., is the city with the most lift-slab buildings, and western Canada generally is more familiar with lift-slab construction than is the East. Toronto has issued four permits for lift-slab buildings, one a 12-story structure, and the area within 100 miles of Toronto has also seen a few lift-slab buildings, but none has so far been erected more than 25 miles east of Toronto. A good many more buildings have been designed for lift slab in Toronto and are awaiting permits, and a lift-slab house is proposed for the Province of Quebec.

In the writer's opinion, lift slab on concrete columns even for high buildings will be improved, and lift slab combined with tilt-up walls will be used more. Lift-slab equipment will likely be used more for large prestressed girders; it has even been used in steel construction.

The system constitutes a valuable new technique and we must learn to use it to its maximum advantage.

Discussion of this paper should reach ACI headquarters in triplicate by Apr. 1, 1958, for publication in Part 2, September 1958 JOURNAL.

Elevated Temperatures of Portland Cement Mixtures Related to Surface Removal*

By R. H. HEISKELL, R. H. BLACK, R. J. CREW, and H. LEE†

SYNOPSIS

The effect of heat on the surface removal of concrete was studied by conducting brush tests on samples which had been subjected to prolonged high temperatures in an oven or in direct contact with an oxypropane flame. Other methods of heat treating surface layers of concrete were investigated by direct application of an oxyaluminum torch on the surface and by the exothermic chemical reactions of pyrotechnic compositions placed directly on the surface.

Tests showed that an oven temperature of 1100 F is required to decompose portland cement compounds sufficiently to reduce the resistance of concrete surfaces to abrasive removal methods. The temperatures produced at the surface of concrete by an oxypropane burner varied, depending on the rate of traverse of the flame, from 200 to 800 F. Spalling of concrete surfaces can be accomplished by prolonged flame treatment with an oxypropane or oxyacetylene burner. Flame treating with a moving oxypropane burner gave a temperature of 120-600 F at 1/64 in. below the surface.

No spalling was observed when an oxyaluminum flame passed over the surface at a speed of 10 ft per min; however, prolonged heating resulted in a rather violent spalling reaction.

INTRODUCTION

In the early stages of the development of radiological decontamination methods, nondestructive liquid cleaning was used on all types of surfaces. Porous concrete surfaces, however, required the removal of a layer of the surface to remove the contaminant. The major effort in the investigation of physical removal methods has been development of the flame treating method. This method consisted of heating the concrete surface with an oxyacetylene or oxypropane flame, then using a floor resurfacing machine to remove the calcined or loosened material. In an effort to improve the efficiency of this operation, information was sought on the effect of elevated temperatures on abrasive resistance of concrete surfaces.

A literature search[1] revealed only information on effect of elevated temperatures on the compressive strength of concrete. The most significant work along these lines was by Miller and Faulkner.[2] The tests reported here were to determine specifically the effect of elevated temperatures on portland cement mixtures in relation to abrasive resistance.

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EXPERIMENTAL DETAILS

The experimental work involved tests on the surface removal of concrete and investigation of methods of calcining surface layers of concrete. The effect of heat on surface removal was investigated by performing brush tests and compressive tests of samples subjected to high temperatures in an oven and on samples subjected to a direct oxypropane flame. Other methods of treating surface layers of concrete were investigated, such as the direct application of an oxyaluminum torch on the surface, and the exothermic chemical reactions of pyrotechnic compositions placed directly on the surface of test cubes.

Test equipment

A concrete surface tester consisting of a weighted wire brush which abraded the surface of a moving test cube was employed to investigate the effect of high temperatures on surface removal operations. Preliminary tests indicated that a weight of 5 lb on the brush and twenty-five 3-in. strokes of the cube at a rate of 60 per min gives a measurable amount of removal; this provided a convenient and reproducible procedure for the studies. To compensate for the grooving action of the wire brush, the cube was rotated 90 deg after 12 strokes. In tests involving the direct application of a flame, the concrete surface tester was equipped with an oxypropane burner which was arranged to direct its flame on the surface of the test cube at a 45 deg angle. The height of the burner was adjusted to bring it within $\frac{1}{8}$ in. of the sample surface. The oxygen and propane lines were connected to rotameters which measured the gas rates. In the flame treating and brushing tests, the test cubes were first flame treated and then brushed.

An oxyaluminum torch was constructed from plans obtained from Temple University.[3] It was claimed[4] that the torch would melt a 3-in. diameter hole through 6 in. of concrete in $1\frac{3}{4}$ min. The torch was arranged to move slowly across a concrete sample. This torch (Fig. 1) used powdered aluminum as fuel with oxygen supporting combustion of the aluminum and propane gas kindling medium. A vibratory feeder metered and fed powdered aluminum to the eductor where it was mixed with the educting flow of oxygen. This mixture passed through the core of the torch head and was ignited by a propane flame from the gas outlets surrounding the primary fuel. The torch was cooled by circulating water through a compartment in torch head.

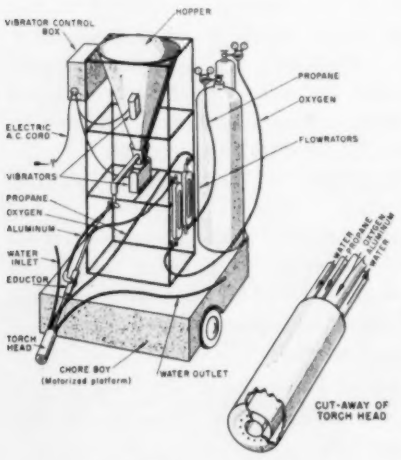

Fig. 1—Oxyaluminum torch

Test samples

Seven-day and 28-day test cubes with a 1½-in. edge were molded of neat portland cement, 1:2¾ portland cement mortar, and a 1:2:3½ portland cement concrete mixture. Mix compositions and the sieve analysis of sand and coarse aggregate are given in Table 1. ASTM specifications C 109-44, C 143-39, and C 192 were met where they were applicable. The samples were cured for 7 and 28 days in accordance with ASTM C 192.

Three groups of test cubes were prepared with 40-gage copper-constantan thermocouples molded in them for measuring the temperature rise resulting from flame treatment. One group was molded of neat cement with the thermocouple bead 1/16 and 1/64 in. below the surface, and another of mortar with the thermocouple bead approximately 1/16 in. below the surface. The third group was also molded of neat cement but had the thermocouple bead at the surface. These samples were cured for 7 days.

Oven tests

All types of test cubes were inserted in an oven maintained at temperatures of 200, 500, 800, 1100, 1300, and 1500 F. These samples were removed from the oven in groups of six after 30-min, 1-hr, 2-hr, and 4-hr heating periods. The temperature of the oven dropped sharply when the samples were inserted (Table 2) and rose gradually until the original temperature was reached and maintained.

Each sample was weighed to the nearest milligram before being placed in the oven; after removal, it was placed in a $CaSO_4$ dessicator to cool and then was reweighed. Compression tests were run on three samples in each group and brush tests were made on the remaining three. In these brush tests a face of the cube smoothed by the mold was used instead of the rela-

TABLE 1—COMPOSITION OF TEST SAMPLES

Mix composition, percent by weight

Sample	Cement	Water	Sand	Gravel
Neat portland cement	75.0	25.0		
Portland cement mortar	23.0	13.8	63.2	
Portland cement concrete	13.6	11.4	27.3	47.7

Sieve analysis, percent by weight

U. S. Sieve	Percentage retained
Sand	
No. 100	98 ± 2
No. 50	72 ± 5
No. 30	2 ± 2
No. 16	0
Coarse aggregate	
No. 12	99 ± 1
No. 8	94 ± 5
No. 4	50 ± 10
⅜ in.	0

TABLE 2—OVEN CONDITIONS AS LOAD IS HEATED

Nominal temperature, deg F	Temperature drop when load was inserted, deg F	Time for oven to reach nominal temperature, min
200	40	15
500	170	38
800	290	27
1100	360	28
1300	435	42
1500	445	45

tively rougher top face. The samples were weighed after the tests to see how much material was removed by the brush.

Flame treating tests

Flame treating tests were conducted on all sample types. For these tests, three gas rates and four treating speeds were used. Four cubes were flame treated, two at a time in the concrete surface tester at each speed and at each of the three gas rates. Following flame treatment, the cubes were cooled to room temperature in a dessicator, then weighed, subjected to the brush test, and reweighed. No compression tests were made on these samples because it was observed in preliminary tests that momentary contact of the flame caused little, if any, change in the over-all compressive strength of the samples.

Samples which had been equipped with thermocouples were given flame treating tests to determine the maximum temperature that could be obtained under each of the three gas rates and four speeds used.

Prolonged flame treating tests were given to the 28-day samples. In these tests the speed of the burner was 2.1 ft per min, and it made two passes over one group of samples and eight passes over the other group. All of the samples were weighed before and after this flame treatment. Then brush tests were made on them and they were again weighed.

Calcining tests

An oxygen-aluminum torch was moved over a concrete slab at different speeds; the minimum speed was 10 ft per min. The oxygen rate was 12 cu ft per min and the propane rate was 1.9 cu ft per min; the aluminum powder was fed at a rate of 100 g per min. Spalling was estimated visually.

Two types of pyrotechnic composition (rapid burning producing much gaseous materials and slow burning, gasless) were prepared and tested on concrete samples which contained thermocouples. One- and 2-g aliquots of these mixtures were spread out evenly on a "thermocouple cube" over a 1 sq in. area. The mixture was ignited and the rise in temperature recorded by a millivoltmeter. Maximum temperatures obtained in preliminary experiments were well below the 1200 F needed to produce appreciable disintegration of cement compounds. Further tests were made with a layer of pulverized asbestos over the pyrotechnic mixture. This procedure resulted in a higher maximum temperature for a longer period.

RESULTS AND DISCUSSION

Oven tests

The results of oven heating on neat portland cement, cement mortar, and concrete are presented in Fig. 2, 3, and 4, respectively. Each point on these curves represents the average of three values. There was considerable variation in the wear and weight loss on the control samples of each group. This variation is believed to be due to slight differences in mixing and molding.

There is an increase in the abrasive resistance of portland cement mixtures that have been heated to 200 F. There is a slight increase in the compressive strength of these mixtures when heated to 500-800 F. Over 500 F the greatest reduction in abrasive resistance occurs during the first hour of heating.

These data show that portland cement compounds decompose sufficiently at slightly higher than 1100 F to show a definite decrease in resistance to

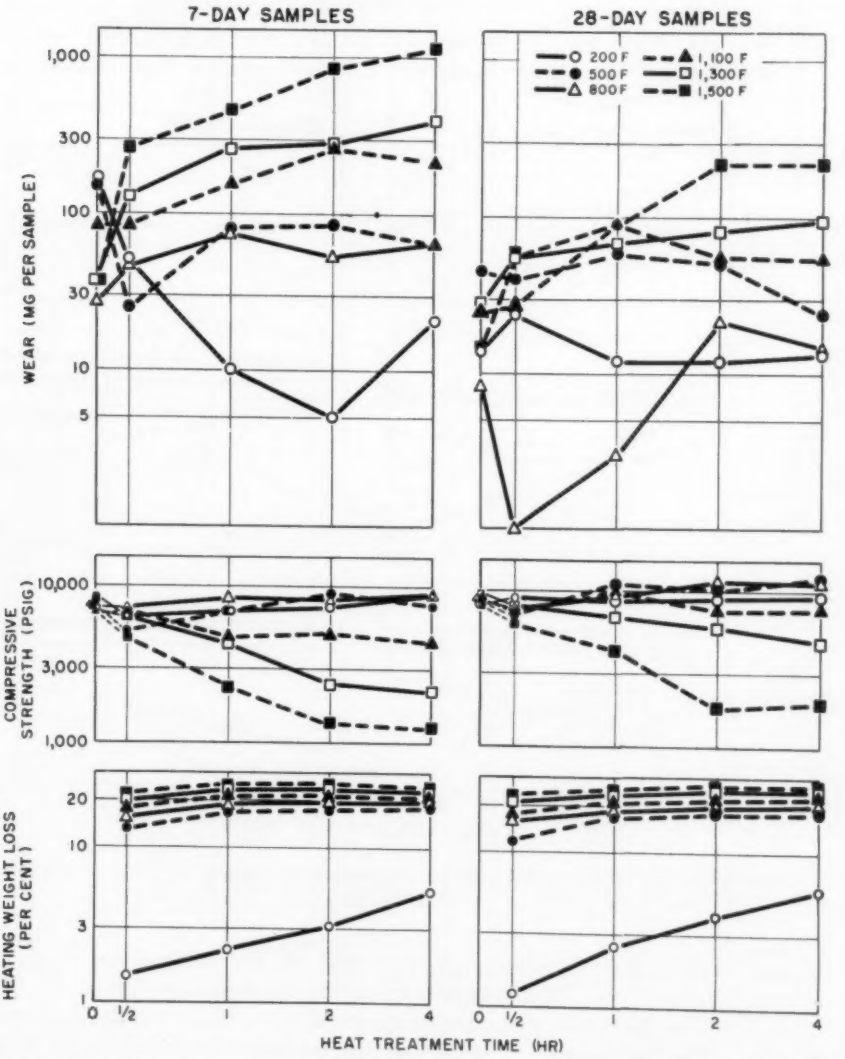

Fig. 2—Oven heating effect on weight loss, compressive strength, and wear of neat portland cement

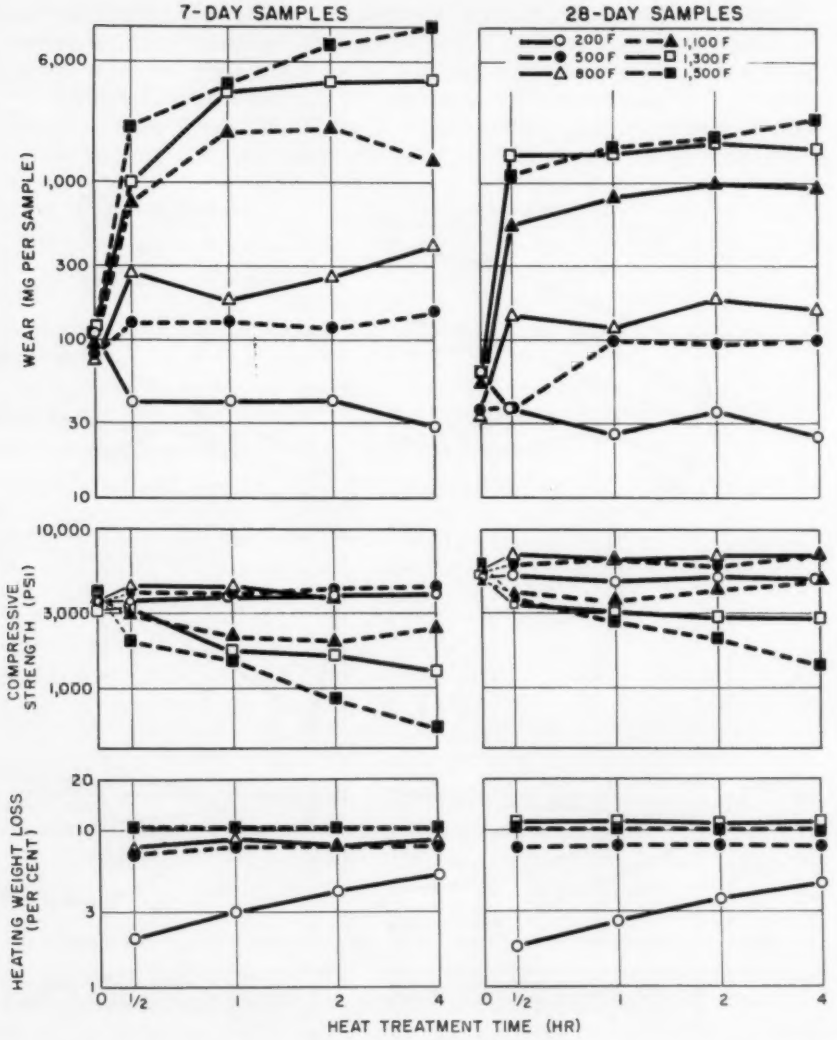

Fig. 3—Oven heating effect on weight loss, compressive strength, and wear of portland cement mortar

surface removal by mechanical means. The 28-day mortar and concrete samples showed only slightly higher resistance to brushing than the 7-day samples. This difference was more pronounced on the neat portland cement samples. The decomposition of cement compounds was shown in all three tests—weight loss, compressive strength, and wear. The amount of surface removed by brushing increased with the rise in temperature from 500 to

1500 F. Wear increased with the length of heat treatment; this increase, however, was very slow after the first hour.

Little change in compressive strength was observed at 200, 500, and 800 F, but at 1100 F a sharp reduction in strength occurred. The compressive strength decreased with increase in temperatures above 1100 F with the greatest reduction at 1500 F, the highest test temperature. These results agree favorably with Miller and Falkner.[2]

The percentage weight loss due to heating increased with each elevation in temperature, but above 500 F the increase was small. It is noted that the

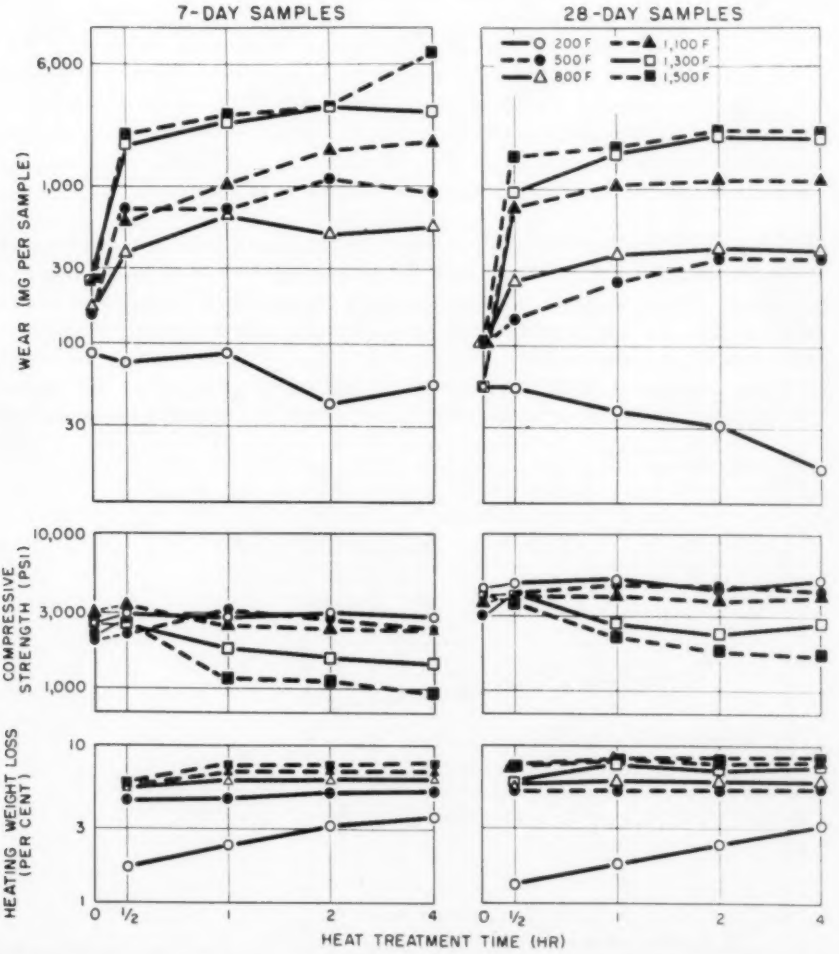

Fig. 4—Oven heating effect on weight loss, compressive strength, and wear of portland cement concrete

TABLE 3—FLAME TREATING TEMPERATURE TESTS*

Rate of pass, ft per min	Gas rate, standard cu ft per min		Maximum temperature, deg F			
			Neat cement			Cement mortar
	Propane	Oxygen	At surface†	0.018 in. below surface†	0.063 in. below surface‡	0.063 in. below surface‡
16.2	0.25	0.83	218	122	83.0	86.7
	0.30	0.92	235	131	85.5	93.7
	0.45	1.22	246	134	87.8	102.2
	0.51§	1.47§	244	138	88.2	
8.1	0.25	0.83	349	196	106.0	131.2
	0.30	0.92	361	209	112.2	134.0
	0.45	1.22	427	242	125.3	142.6
	0.51§	1.47§	404	208	114.5	
4.6	0.25	0.83	458	271	132.3	140.0
	0.30	0.92	469	237	121.2	151.0
	0.45	1.22	482	264	130.2	165.0
2.1	0.25	0.83	722	444	189.0	218.2
	0.30	0.92	793	518	197.3	241.5
	0.45	1.22	813	606	227.0	307.3
	0.51§	1.47§	1135	778	297.3	

*Room temperature 64 F.
†Average of three values.
‡Average of six values.
§Unstable flame-gas rates too high for satisfactory flame propagation.

weight loss at temperatures from 500 to 1500 F reached equilibrium after approximately 1 hr of exposure, but the percentage loss is greater at each of the higher temperatures. Apparently more decomposition occurs at higher temperatures, but the degree of decomposition at each temperature reaches a maximum after exposure for approximately 1 hr. This is also approximately the time required to heat the oven to control temperature after insertion of the samples. At 200 F, however, the loss in weight continued to increase with

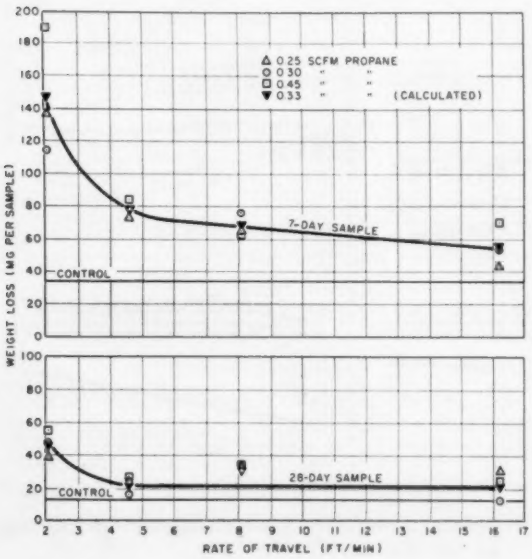

Fig. 5—Flame treating effect on wear of neat portland cement

time of exposure. As would be expected, the weight loss of the three different compositions was approximately proportional to the percentage of portland cement present. The percentage weight loss on the 28-day samples of each composition was almost identical to that for the 7-day ones.

Flame treating tests

The temperatures to which the blocks were subjected under various conditions of flame treating are presented in Table 3. It will be observed that the maximum temperature obtained was 1135 F at the surface of the block. At approximately 1/32 in. below the surface the temperature was only 778 F and at 1/16 in., 297 F. The operating speed at which these temperatures were obtained was 2.1 ft per min. The temperatures obtained at speeds of 16, 8, and 5 ft per min ranged from 200 to 500 F at the surface and from slightly above room temperature to 130 F at the 1/16-in. depth. These temperatures are far below those needed to break down portland cement compounds, as found in the oven tests. It will be noted that the temperatures obtained for the mortar samples were higher than those for neat cement. Spalling occurred only at the lowest operating speed (2.1 ft per min). The amount of spalling was inconsistent and varied considerably on different samples. By comparison, there was practically no spalling observed during the oven tests, even at 1500 F.

Data for surface removal by brushing after flame treatment are presented in Fig. 5, 6, and 7. Even though the temperatures obtained by flame treat-

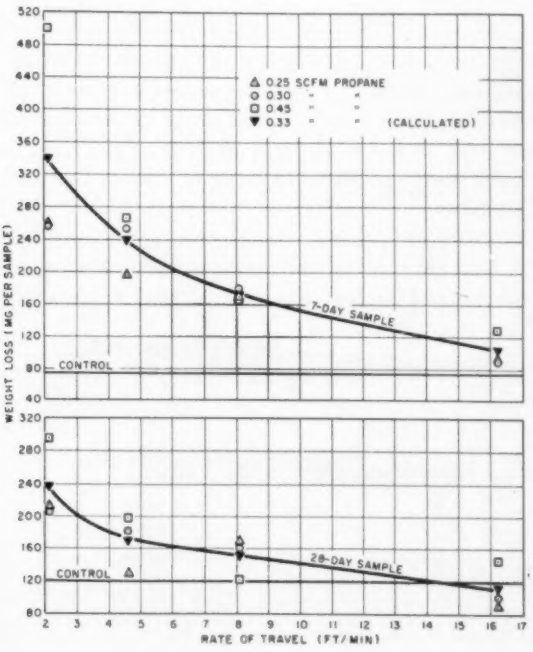

Fig. 6—Flame treating effect on wear of portland cement mortar

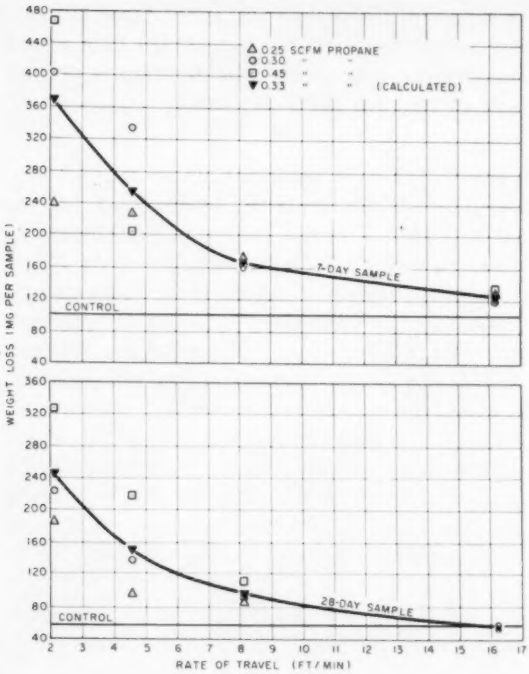

Fig. 7—Flame treating effect on wear of portland cement concrete

ment are relatively low, some decomposition of the surface was noted in the brush tests. The amount of surface removed at each of the flame speeds did not appear to increase with increase in gas rate, except at the lowest speed. A decrease in the speed of flame treatment gave a corresponding increase in the amount of surface removed on both the mortar and the concrete samples, but this increase was not as pronounced on the neat cement samples.

It is observed that the amount of surface removed after flame treating was at least as great as that removed after the oven heating method, even though the time of heating was only a fraction of that in the oven.

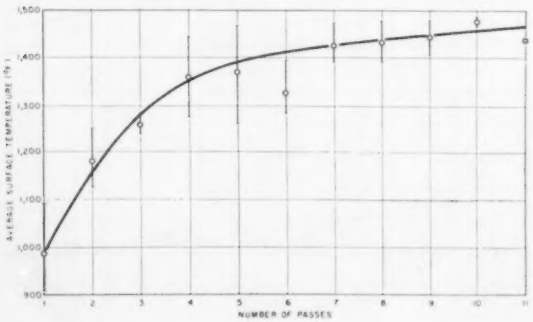

Fig. 8—Multi-pass flame tests on 28-day neat portland cement samples (Thermocouple at surface; rate of travel = 2.1 ft per min; gas rate = 0.45 standard cu ft per min propane)

TABLE 4—MULTI-PASS FLAME TEST*

Pass No.	Surface temperature,† deg F	Weight loss, g					
		Neat cement		Cement mortar		Cement concrete	
		Heating	Wear	Heating	Wear	Heating	Wear
0			0.031				0.058
1	988						
2	1179	1.883	0.121	1.929	0.909	0.985	1.443
8	1431	6.127	0.449	4.011	2.614	4.064	2.799

*The gas rates were propane at 0.45 and oxygen at 1.22 standard cu ft per min. The rate of flame travel was 2.1 ft per min. The flame was passed over the samples tested every 15 sec throughout the tests.
†The surface temperatures obtained with the other passes are: pass No. 3, 1254 F; 4, 1358 F; 5, 1369 F; 6, 1323 F; 7, 1425 F; 9, 1443 F; 10, 1475 F; and 11, 1435 F.

Repeated passes of the burner over the same block gave steadily rising temperatures, as shown in Table 4. The results of tests in which both two and eight passes of the flame were made over the samples are presented in Fig. 8 and 9. Temperatures of 1180 and 1430 F were obtained on the thermocouple blocks under these conditions. Results of the brush tests indicated that the direct flame gave greater decomposition of the cement compounds than oven heating at these temperatures. There was a great deal of spalling during these repeated flame tests, which, together with evaporation of water, accounts for the loss of weight from heating.

Calcining tests

No spalling occurred when the oxyaluminum torch was moved across 1-ft square concrete samples at the minimum test speed of 10 ft per min.

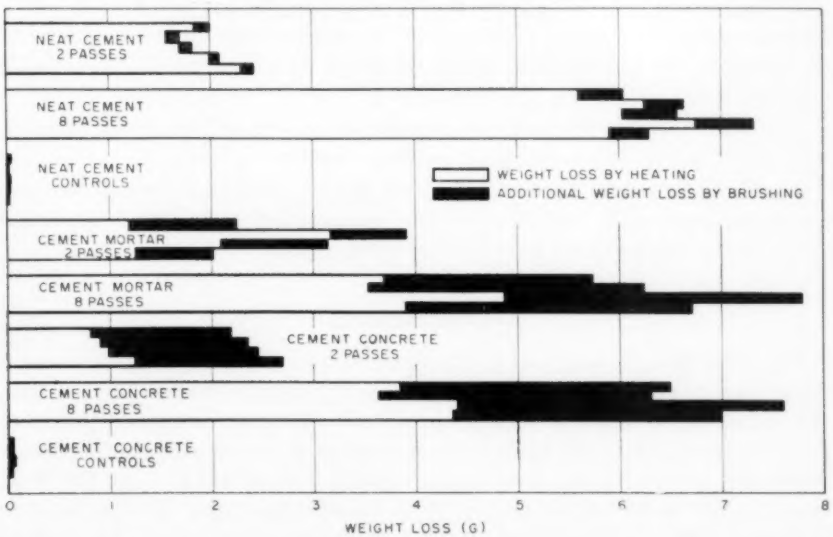

Fig. 9—Multi-pass flame treating effect on weight loss of portland cement samples

TABLE 5—PYROTECHNIC TEST

Mixture No.	Composition, percent by weight	Heat of combustion, cal per g*	Amount used, g	Temperatures, deg F	
				Surface	1/16 in. below surface
	Rapid-burning mixtures				
BM	90 percent BaO_2; 10 percent Mg	460	1	558	178
BI-2	80 percent Bi_2O_3; 20 percent Mg	1111	1	338	
BI-11	50 percent Sb_2O_3; 50 percent Mg	1546	1	298	112
	Gasless mixtures (slow-burning)				
DI-149	75 percent BaO_2; 25 percent Sb_2S_3	240	1	715	170
DI-149	75 percent BaO_2; 25 percent Sb_2S_3		2	825†	
DI-417	70 percent BaO_2; 30 percent Mn	485	1	548	228
DI-417	70 percent BaO_2; 30 percent Mn		2	796	
DI-417	70 percent BaO_2; 30 percent Mn		2	1086†	
DI-452	70 percent BaO_2; 30 percent Se	165	1	462	155
DI-470	50 percent Sb_2O_3; 50 percent Mn	405	1	530	194
DI-470	50 percent Sb_2O_3; 50 percent Mn		2	705†	
DI-486	70 percent Bi_2O_3; 30 percent Mn	305	1	608	154
DI-486	70 percent Bi_2O_3; 30 percent Mn		2	1008†	

*See References 8 and 9.
†Layer of pyrotechnic covered by blanket of asbestos.

It was necessary to direct the torch on one spot for approximately 5 min before spalling did occur. Upon spalling, however, a large area approximately 8 sq in. blew off, leaving a cavity about ½ in. deep.

In 1 hr of operation at the test rate, 220 cu ft of oxygen and 13 lb of aluminum powder were consumed to remove a surface area of approximately 3 sq ft.

Results of the pyrotechnic tests are presented in Table 5. It will be observed that the gasless or slow burning mixtures gave the highest maximum temperature even though two of the rapid burning mixtures have much higher heats of combustion. The lower temperature results because heat is lost to the atmosphere in a flash. The blanket of asbestos sealed in more of the heat generated by the gasless compositions as shown by comparison between results of Tests 7 and 8.

The residue from the gasless compositions glowed red for several seconds and kept the block surface hot for several minutes.

A maximum temperature of over 1000 F was obtained in two of the tests conducted and, with variations in the composition used, this temperature can possibly be increased several hundred degrees.

CONCLUSIONS

The oven tests showed that a temperature in excess of 1100 F is required to decompose portland cement compounds sufficiently to reduce substantially the resistance of concrete surfaces to abrasive removal methods. The temperature produced at the surface of concrete by an oxypropane burner varies, depending on conditions, from approximately 200 to 800 F when the flame moves across the surface at a rate of 16 to 2 linear ft per min. The temperature at about 1/64 in. below the surface was found to be approximately 120 to 600 F under the same conditions.

It can be concluded from these tests that flame treating has little effect on the decomposition of concrete surfaces except when the flame is in contact and moves at 2 ft per min or less; decomposition at this flame travel rate is very shallow. The slow operating speed required to produce a satisfactory rise in temperature limits the use of the flame treating method of decontamination to relatively small areas.

Spalling of concrete surfaces can be accomplished by prolonged flame treatment with an oxypropane burner, but the time required and the inability to control this spalling makes this procedure impractical as a means of removing a thin surface layer.

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Lightweight Structural Concrete Proportioning and Control*

By GEORGE H. NELSON† and OTTO C. FREI‡

SYNOPSIS

Physical properties and gradation of an expanded shale aggregate produced in Georgia are given. Effect of fine aggregate:coarse aggregate ratio and effect of entrained air on properties of lightweight structural concrete are discussed. Recommends optimum air content for maximum strength of lightweight concrete with from 3 to 9 sacks of cement per cu yd. Proposes a proportioning method which makes use of a *specific gravity factor*. Using this proportioning method with aforementioned aggregate, tests were made on a series of mixes with from 3 to 9 sacks of cement per cu yd. Properties reported include compressive, flexural, and bond strength; modulus of elasticity; diagonal tension; and thermal conductivity. Field control practices for lightweight structural concrete are briefly considered.

The purpose of this paper is to outline experiences with Georgia lightweight aggregate concrete, to offer test data and conclusions for comparison with those of earlier authors, and to propose methods which might further simplify the design and control of lightweight structural concrete.

PHYSICAL PROPERTIES OF AGGREGATE

The aggregate is a local expanded shale produced by burning properly sized raw shale in rotary kilns at a temperature of 2000 F, then crushing the cooled material, and screening into three separate sizes which are stored individually.

The finished aggregate is available for shipment in separate sizes for batching at the concrete plant or job site, or can be blended and mixed at the plant in any proportion required. This final assembly of particle sizes from the previously separated size groups assures uniformly accurate, full-range gradation of aggregate for concrete making purposes.

Gradation

The fineness modulus and unit weight of the three separate sizes are as shown in Table 1. Gradations of the separate sizes of this aggregate are in accordance with ASTM C 330-53T and C 331-53T. The blending and mixing

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TABLE 1—FINENESS MODULUS AND UNIT WEIGHT OF THE THREE SEPARATE AGGREGATE SIZES

Size range of particles	¾-⅜ in.	⅜-3/16 in.	3/16 in. –dust
Fineness modulus	7.14	5.37	3.13
Weight, lb per cu ft, dry jigged	46.1	50.5	56.2

TABLE 2—CRUSHING STRENGTH OF AGGREGATE AT 1-IN. COMPACTION

Percent aggregate		Crushing strength, psi
Fine	Intermediate	
20	80	1598
30	70	1768
40	60	2178
50	50	2291
60	40	3593
70	30	3310
80	20	3211

facilities of the producer can assure uniform gradation of aggregate to cover any possible concrete requirements within limits indicated.

Mortar strength

Mortar strength tests conducted on the fine aggregate show a development of 115 percent of the strength of standard Ottawa sand at 28 days, well above the ASTM requirements.

Soundness

The loss from five-cycle sodium sulfate tests for soundness, conducted in accordance with ASTM C 88-46T was 1.22 percent for the coarse and 1.16 percent for the fine aggregate, both being well within the ASTM requirements for heavy aggregates.

Crushing strength

Crushing strength tests were conducted on the aggregate itself in accordance with general practice for crushing test at 1 in. of compaction. The crushing strength of aggregate has, for a number of years, been given considerable attention, and the relative concrete making qualities of aggregates have at times been predicted on this basis. The writers cannot as yet agree with this method of aggregate evaluation, as it is felt that the test procedure lacks refinement and that the test results are influenced too greatly by the particular gradation of the aggregate under test.

Crushing strength tests were accordingly conducted to determine the effect of gradation on crushing strength. Samples contained varying ratios of fine to medium aggregate and the strengths obtained are outlined in Table 2. All of the strength results obtained on the same aggregate cannot be correct. It does appear, however, that an optimum grading exists for maximum crushing strength, beyond which point a decrease in crushing strength occurs. It likewise appears possible that further investigation along these lines may lead to the development of a rapid test for establishing the proper ratio between fine and coarse aggregate for maximum strength of lightweight concrete.

Until such further studies are completed, or until the method is qualified as to gradation, the writers do not consider the crushing strength of the aggregate significant for mix proportioning or for comparison of aggregates of the same general class.

EFFECT OF MATERIALS ON MIX PROPORTIONING

With the various physical characteristics of expanded shale aggregate in mind, one need only to apply the usual principles of good concrete proportioning to successfully produce consistent and high quality lightweight structural concrete.

Fine-to-coarse aggregate ratio

The sharp, angular nature of the crushed expanded shale naturally calls for a higher percentage of fines (passing the No. 4 sieve) than is the case for sand and gravel. This change is not unexpected inasmuch as it is no different than is the case with sharp, angular sand and crushed rock in heavy concrete. Most published information on fine-to-coarse proportioning indicates that approximately 50-65 percent by volume of the total aggregate should pass the No. 4 sieve. Such proportioning appears about right for average structural strength requirements. The actual ratio of fines to coarse aggregate, is however, subject to considerable variation for best results, depending upon the end result regarding concrete weight and compressive strength desired.

Good workable concrete can be proportioned with almost no fine aggregate and in other cases with all fine aggregate. This is not new or original but depends upon the type of concrete required and the amount of air entrainment used.

Effect of entrained air

The ratio of fine to coarse aggregate to be used in a specific mix is directly related to the entrained air content, which in turn is dictated by the weight of concrete that is desired, the degree of concrete economy required, and the compressive strength essential to the particular application. Within the range of 0-30 percent entrained air, it is possible to obtain almost any desired weight of concrete, bearing in mind that compressive strength decreases as air content increases. Extremely light concrete weights for this type of aggregate in the range of 70-85 lb per cu ft are readily available with comparatively high entrained air, but maximum air content is limited where high structural strength of concrete is paramount.

For structural lightweight concrete the percentage of fine aggregate should, therefore, be the lowest possible amount which can be used with the lowest possible entrained air content to produce required workability. This, as can be expected, will result in the highest compressive strength at any given cement factor inasmuch as the air content is minimized and total mixing water is reduced due to the low percentage of fines in the mix. An actual deficiency of fines should be avoided, however, because the fine material has somewhat greater density than the coarse aggregate and consequently is important for attaining strength of concrete.

The absolute minimum percentage of fine aggregate might well be expressed as that at which harshness begins to develop in the mix at the air content selected. The absolute maximum percentage of fine aggregate might be

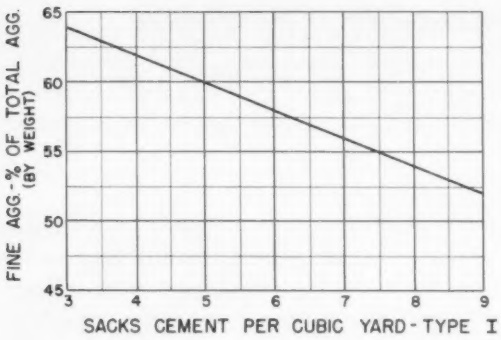


Fig. 1—Optimum proportion of fine aggregate for given cement content

expressed as the point where additional mixing water is required for desired slump at the same air content. Generally speaking, it has been found that 50-65 percent by weight of aggregate passing the No. 4 sieve is approximately correct to cover the range of 3 to 9 sacks of cement per cu yd of concrete.

As noted in Fig. 1, it was found that the fine aggregate percentage should decrease about 2 percent for each sack increase in cement content.

Recommended entrained air

Using fine aggregate ratios within this range, we have developed, through experience, a recommended entrained air content for maximum strength of concretes ranging from 3 to 9 sacks of cement per cu yd. This is shown in Fig. 2 with corresponding compressive strength in Fig. 3. These concretes range in slump from 2 to 4½ in. and are extremely workable. Due to the lighter weight of expanded shale concrete results of the slump test are not directly comparable to those for heavy concrete, and actually this type of concrete has equivalent workability with 2 to 3 in. less slump than is required of regular weight concrete. Following the usual proportioning principle, extra fluidity of mix requires more water which in turn requires more cement to maintain the desired strength.

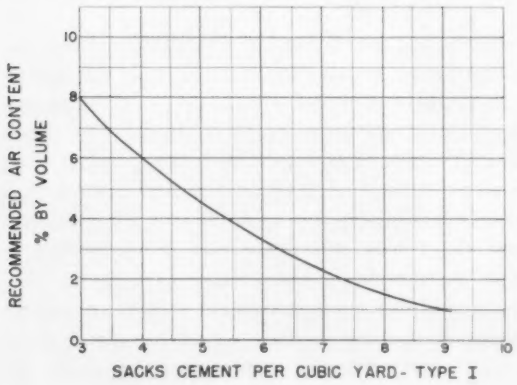


Fig. 2 — Recommended air content for a given cement factor

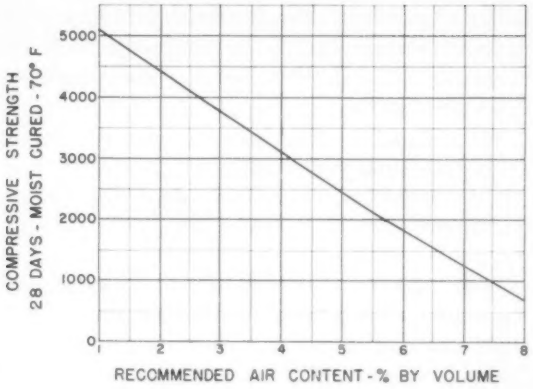


Fig. 3—Compressive strength of mixes recommended in Fig. 2

Note that the air content ranges from 8 percent at 3 sacks per cu yd, down to 3½ percent for 6 sacks per cu yd, and down to 1 percent for 9 sacks per cu yd. This increase in air content as the cement content decreases is solely to achieve desired workability and is not to be regarded as an aid to durability as is often the case with heavyweight concrete. Expanded shale concrete can be extremely durable and much published information is available on this subject.

Method of air determination

Difficulties were encountered in attempting to determine the entrained air content of the mixes; and the gravimetric method, the method utilizing the differential in air content between air-entraining and non-air-entraining concrete, and the pressure methods were tried and discarded. The volumetric (rolling) method provided the most consistent and reproducible results.

The amount of air-entraining admixture required was found to be about twice the normal amount used for heavyweight concrete for cement factors below 6 sacks per cu yd. At cement factors of 6 sacks and above the amount was found to be about the same or less than for heavyweight concrete.

GENERAL MIX PROPORTIONING METHODS

There are two methods generally in use for proportioning lightweight concrete. The first requires that the total volume of aggregates needed to produce 1 cu yd of concrete on a moisture-free basis be determined. This moisture-free volume is then converted to pounds for the basic design weights, these weights being corrected for such moisture as may be contained in the stockpiled aggregates. Mixing water is added by trial until the proper slump is obtained. This method assumes that, once the weight per cu ft of each of the aggregates is determined on a dry loose basis, no appreciable variation in gradation or specific gravity occurs, and that this fixed weight will continue to contribute the same solid volume to each batch of concrete.

Investigation of this method led to the conclusion that variations in concrete yield as affected by either changes in gradation or specific gravity of the aggregate or both were beyond those accepted as normal in heavyweight concrete practice.

The second method requires the determination of the absorption and saturated, surface-dry specific gravities of the fine and coarse aggregates. These values are used to calculate the solid volumes of the aggregates for design purposes as is customary in the design of conventional heavyweight concrete.

It is difficult, if not impossible, to determine the point at which the fine aggregate is actually saturated and surface-dry. The coarse aggregate presents less difficulty. This leads to the opinion that the values obtained for absorption, rate of absorption, and bulk specific gravity cannot be relied upon for proportioning until improved and more reliable test methods are developed. This method also assumes that the bulk specific gravity value so obtained and used to calculate solid volume will remain constant. The writers have found that this is not the case, and variations in concrete yield by this method are also beyond acceptable tolerances.

It was considered advisable that some more accurate basic procedure be established. Such a procedure should, if possible, circumvent the problems caused by uncertainties in connection with absorption, rate of absorption, and bulk specific gravity.

PROPOSED PROPORTIONING METHOD

In all preliminary concrete proportioning, trial batches provide initial knowledge of the fundamental characteristics of any concrete. It is possible to establish proper proportions of fine to coarse aggregate, required cement factor, proper percentage of entrained air, and total mixing water required only after sufficient trial mix data have been assembled to provide this information.

Based on the knowledge gained through trial mixes, the known factors in our approach to lightweight concrete proportioning are, therefore, aggregate proportioning (fine-to-coarse), required cement factor, recommended entrained air content, and the proper amount of total mixing water to produce the required slump. Unknown factors are: the total absorption of the aggregate in the concrete during mixing, transporting, and placing; the rate of this absorption; and the true bulk specific gravity of the aggregates. These unknown factors may be temporarily neglected.

Proportioning example

Assume that a lightweight structural concrete with 3000 psi compressive strength and a 3-in. slump is required. From the known factors, it is determined that 6 sacks of cement per cu yd will be required, that 3.5 percent of entrained air is needed, and that 64 gal. total mixing water will be necessary per cu yd to produce the desired 3-in. slump. We do not know the exact

total weight of dry aggregate that will be necessary to produce exactly 1 cu yd of concrete since the unknown factors enumerated above are not reliably measurable.

The absolute volumes of the known quantities of cement, air, and total water can be accurately calculated and totaled as follows:

Cement, 6 sacks	2.892 cu ft
Air, 3½ percent	0.945
Water, 64 gal.	8.533
	12.370 cu ft

The combined absolute volume of these three ingredients is 12.37 cu ft; 14.63 cu ft of additional solid volume is required to provide 1 cu yd of concrete. Therefore, the dry aggregate to be added must provide the 14.63 cu ft of solid volume plus an additional solid volume equal to the unknown quantity of water which will be absorbed by the aggregate from the total given mixing water.

If we now assume that the aggregate has no absorption, (as is the case with dense, heavyweight aggregates in a saturated, surface-dry condition) and we have determined the specific gravity of the aggregates, it is simple to calculate the weight of aggregate required and complete the mix.

The problem with lightweight aggregate, however, is not so simple. In the first place, some of the total water, which we have calculated as absolute volume, will be absorbed by the aggregate, reducing the yield contribution of the water. How much water will be absorbed we have been unable to determine accurately since the absorption varies with production, the amount of water initially present in the aggregate, and the inaccuracies of testing methods.

Secondly, we do not know accurately what the true bulk specific gravity of the aggregate may be, since the determination is inaccurate and doubtful.

Trial mix

We can, however, determine the moisture of the predampened aggregates at hand, and know the percent of water and the percent of dry aggregate represented per lb of damp material. We also know, from previous trial mixes, that approximately 1770 lb of dry aggregate is required per cu yd of concrete. We will begin with 1750 lb. We can, therefore, weigh the proper amount of damp aggregate that will provide the 1750 pounds of dry material, and calculate the weight of water that this amount will contribute to the total mixing water.

Then put into the mixer the 64 gal. of water, less the amount carried by the aggregates. It is also advisable to withhold several gallons more to enable final adjustments to the mix for accurate slump attainment. Assume for this example that 167 lb (20 gal.) of water is carried by the damp aggregates. Therefore add to the mixer 64 gal., less 20 gal., less an additional 5 gal. for final adjustments, or a temporary total of 39 gal. Add the air-entraining agent to this water, allowing a few moments mixing time for dispersal throughout the water.

Next introduce the weighed damp aggregates, allowing at least 1 min mixing for absorption of such amount of the mixing water as the aggregates may still require.

The weighed amount of cement is then added, allowing ample time for complete mixing, and the withheld 5 gal. of water is added as necessary to produce approximately the desired 3-in. slump.

When the mix is completed, the record shows the total weight of bone-dry aggregate, the weight of cement, the amount of air-entraining agent, and the weight of total water in the mix.

Assume that the following data were obtained:

Slump	2¾ in.
Total water	63 gal.
Air content	3½ percent
Dry aggregate	1750 lb
Weight of batch	2830 lb
Unit weight of concrete	106 lb per cu ft
Yield of concrete	26.7 cu ft

The volume of the known factors can be calculated as follows:

564 lb cement	2.892 cu ft
63 gal. water	8.40
3.5 percent air	0.93
Total	12.222 cu ft

Volume contributed by aggregate and unknown factors = 26.70 − 12.22 = 14.48 cu ft

Substituting the above figures in the familiar specific gravity formula:

Lb aggregate = Absolute volume of aggregate × specific gravity × 62.4
1750 = 14.48 × specific gravity × 62.4

the specific gravity in this case is found to be 1.94. Since this is not a true specific gravity, the writers have designated this figure the *specific gravity factor* for the aggregate in this mix.

This *specific gravity factor* represents the bulk specific gravity which would have had to be used to determine the actual weight of dry aggregate to produce 26.7 cu ft of concrete in order to allow for the loss in water which was absorbed by the aggregate. This replaces the bulk specific gravity as normally used in heavyweight concrete design, and in effect, reduces all the variables to a constant. It represents a figure that results in the net absolute yield of the aggregates no matter what their percentage of absorption or their true specific gravities may be.

Data from this trial batch can then be used to proportion the originally contemplated 3000-psi mix. One additional gal. of water is estimated to be added to obtain a 3-in. slump. The amounts of each ingredient are then calculated as follows:

564 lb cement	2.892 cu ft
64 gal. water	8.533
3.5 percent air	0.945
	12.370 cu ft

27.000 cu ft
−12.370
14.630 cu ft

$14.36 \times 1.94 \times 62.4 = 1770$ lb

An actual trial batch using 6 sacks of cement, 64 gal. total water, 1770 lb of dry aggregate and an air content of 3.5 percent will result in exactly 27 cu ft of concrete with a cement factor of exactly 6 sacks per cu yd.

The specific gravity factor can be calculated in moments from any laboratory or field batch in which known weights of materials are used, and on which concrete an air and unit weight determination is made. It thus provides an immediate means of adjustment or correction to any field change in specific gravity or absorption of the aggregates. It likewise provides the same degree of control for special concretes where weight requirements may call for a mixture of light and heavy aggregates.

Fig. 4 shows the change in weight of total dry aggregates per cu yd of concrete that is required for proper yield as the specific gravity factor varies from 1.85 to 2.20. The weight of material required increases as the specific gravity factor increases.

This chart is used in field control and where changes in either absorption or specific gravity of the aggregates, or both, have resulted in a noticeable change in yield. After determining the specific gravity factor from a sample of the concrete, a rapid referral to the chart immediately signifies the exact change in aggregate weight that is necessary to provide a return to accurate yield of concrete.

For example, assume that a change in yield has occurred in the field using a specific gravity of 1.94. The yield is short. Determination of specific gravity factor from the concrete at hand shows a new figure of 2.00. Follow-

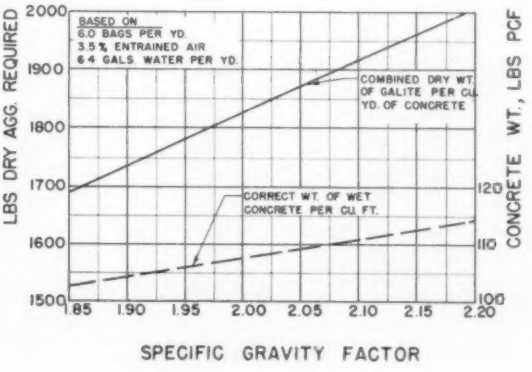


Fig. 4—Weight of total dry aggregates required per cu yd, for varying specific gravity factor

ing the line from the value of 2.00 we find that the weight of total dry aggregate should be increased to 1825 lb. This will result in a return to exact and proper yield with an increase in wet concrete weight from 106 to 107.5 lb per cu ft.

TESTING PROGRAM AND PROCEDURES

Tests were conducted on a series of mixes from 3 to 9 sacks of cement per cu yd in 1-sack increments. The purpose of the program was to develop data for use by architects and engineers. Properties of the seven test mixes are given in Table 3.

All aggregate was obtained from thoroughly mixed stockpiles and was used in a partly saturated condition in order to simulate field conditions. A stationary 2-sack mixer was used in order to provide a batch large enough to mold all types of test specimens from the same batch of concrete. Most of the water and all of the air-entraining agent were placed in the mixer. The aggregate was added and allowed to mix 1 min. The cement and additional water were then added and mixing continued 4 min. Since many specimens

TABLE 3—CHARACTERISTICS OF MIXES IN THE TEST PROGRAM

Mix No.	Cement factor, sacks per cu yd	Total W/C ratio.* gal per sack	Slump, in.	Total air content,† percent by volume	Air-entraining agent, oz per sack	Weight, lb per cu ft			24-hr absorption, lb per cu ft
						Wet	Air-dry‡	Oven-dry‡	
1	3.18	21.3	3.0	8.0	2.0	96.2	88.5	79.7	13.5
2	3.94	16.2	3.0	8.0	1.25	98.5	92.0	83.7	13.4
3	4.93	13.2	2.5	6.5	1.25	100.4	97.2	87.0	13.2
4	5.95	11.3	3.0	4.0	1.0	104.0	99.5	89.0	13.2
5	6.82	10.1	2.0	3.0	0.75	108.3	104.0	92.5	13.8
6	7.62	9.6	2.75	3.5	0.5	108.0	105.0	93.5	12.9
7	9.34	7.3	4.25	1.5	None	111.1	107.7	96.8	12.3

NOTE: All data based on averages of three specimens.
*Total W/C ratio includes water absorbed by aggregates.
†Air content determined by "Roll-a-meter" method.
‡Air-dry and oven-dry weights determined at 28 days.

TABLE 4—RESULTS OF TESTS

Mix No.	Cement factor, sacks per cu yd	Compressive strength,* psi				Flexural strength*, psi 28 days	K factor,† Btu per sq ft per hr per deg F per in.	Tangent modulus of elasticity* E	Bond strength‡ ASTM A 305-50 T Bars 9-in. embedment			Diagonal tension shearing stress,‡§ psi
		7 days	28 days	6 mos.	1 year				Vertical	Top horizontal	Bottom horizontal	
1	3.18	475	960	1573	1845	200	2.44	1,240,000	276	149	—	—
2	3.94	1020	1631	2363	2633	360	—	1,670,000	582	380	399	—
3	4.93	2046	2875	3847	3953	385	2.66	1,760,000	877	823	863	—
4	5.89	2506	3372	4280	4467	411	—	1,960,000	1175	880	863	—
5	6.82	2869	4256	4800	5420	431	3.15	2,225,000	1104	896	1052	—
6	7.62	3300	4642	4917	5895	480	—	2,320,000	1118	1123	1108	—
7	9.34	3775	5210	5370	6013	534	3.38	2,440,000	1226	—	1179	—
8	—	—	2300	—	—	—	—	—	—	—	—	164
9	—	—	4800	—	—	—	—	—	—	—	—	231
10	—	—	6400	—	—	—	—	—	—	—	—	260

*Compressive and flexural strengths and modulus of elasticity tests are the averages of three specimens, moist cured at 70 F.
†K factor is average of two specimens, moist cured at 70 F for 28 days, then dried to constant weight.
‡Bond strength and diagonal tension tests are average of two specimens moist cured at 70 F.
§Diagonal tension tests conducted on separate mixes.

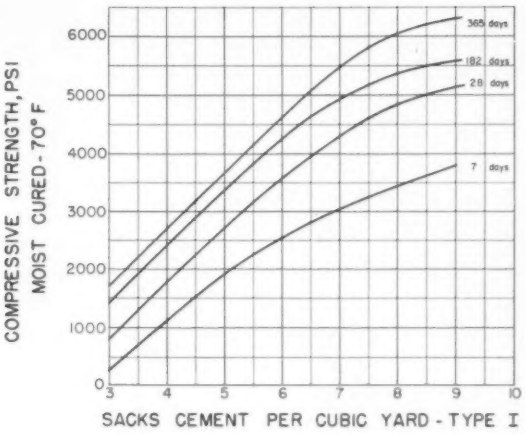


Fig. 5—Relation between compressive strength and cement content

had to be molded mild agitation was used intermittently during the molding period. All specimens were fog cured at 70 F until tested. Type I portland cement was used in this series.

Using the above method of proportioning, the following properties were tested: compressive and flexural strength; vertical and horizontal bond strength; modulus of elasticity; shear in diagonal tension; and thermal conductivity. Test results are summarized in Table 4.

Compressive strength

Tests were conducted on 6 x 12-in. moist cylinders. Additional cylinders were made for the air-dry and oven-dry weight determinations. Tests were made at 7 days, 28 days, 6 months and 1 year.

The relationship between compressive strength and cement content is shown in Fig. 5. The 28-day compressive strengths ranged from 800 psi at 3 sacks per cu yd to 5200 psi at 9 sacks. The gain at later ages appeared to be normal.

The relationship between the weight per cu ft of the concrete, wet, air-dry, and oven-dry, and the cement content are shown in Fig. 6. The same data

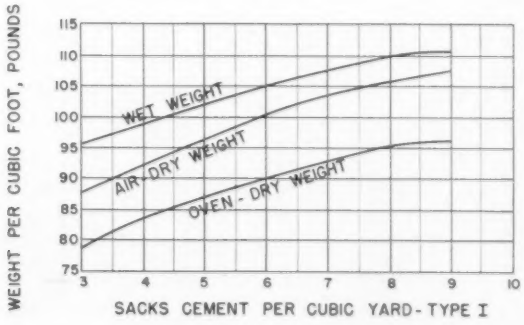


Fig. 6—Relation between weight of concrete and cement content

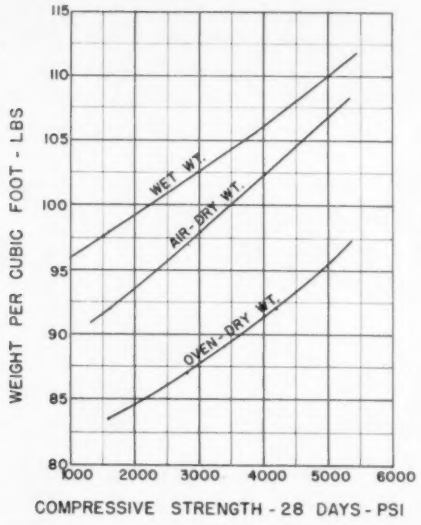


Fig. 7—Relation between weight and compressive strength

are plotted in Fig. 7 which expresses the relationship of weight and 28-day compressive strength.

Over the range of cement contents the wet weight of the concrete varied from 96 lb per cu ft at 3 sacks to 111 lb per cu ft at 9 sacks. The air-dry weights were from 3 to 8 lb per cu ft lighter, the richer mixes tending to retain their original moisture longer. Wet concrete weighing 98.5 lb per cu ft developed 2000 psi; concrete weighing 102.5 lb per cu ft developed 3000 psi; and concrete weighing 110 lb per cu ft developed 5000 psi. The air-dry weight of 2000-psi concrete was 93.5 lb per cu ft; for 3000-psi concrete, 98 lb per cu ft; and for 5000-psi concrete, 107 lb per cu ft.

Flexural strength

Flexural strength tests were conducted on 6 x 6 x 30-in. moist beams in accordance with ASTM C 78-49. The relationship between the modulus of rupture and the compressive strength is depicted in Fig. 8. The 2000-psi concrete developed 320 psi flexural strength, the 3000-psi concrete developed 400 psi, and the 5000-psi concrete reached 515 psi.

Bond strength

Vertical and horizontal bond strength tests were conducted in accordance with ASTM C 234-54 using 9-in. embedment and the new A 305 50-T bars.

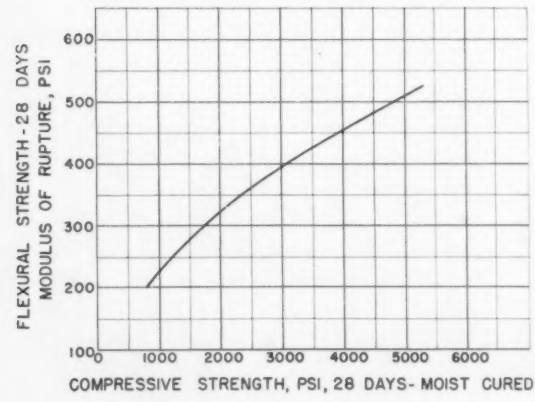


Fig. 8—Relation between modulus of rupture and compressive strength

Results of these tests are shown in Fig. 9, illustrating the relationship between vertical or horizontal bond and the compressive strength. The 3000-psi concrete developed vertical bond strength of 900 psi and a horizontal bond strength of 840 psi. Vertical bond strengths ranged from 400 psi at 1000 psi compressive strength to 900 psi at 3000 psi, and to 1180 psi at 5000 psi.

Modulus of elasticity

Static modulus of elasticity was determined on selected test cylinders by using a compressometer attached to the specimens. Data expressing the relationship between E (tangent modulus) and compressive strength are plotted in Fig. 10. These values range from 1,240,000 at 1000 psi to 1,850,000 at 3000 psi, and up to 2,450,000 at 5000 psi.

Diagonal tension

Resistance of reinforced lightweight concrete beams to diagonal tension was investigated in a series of tests on beams having no web reinforcing.

Beams of 8 x 18-in. cross section were used with a test span of 66 in. One #7 straight round deformed bar meeting ASTM A 305-47T was used as tension reinforcing.

Beam specimens were cast in fabricated wood forms, removed from the forms in 2 days, and cured in a moisture room until the time of test. Concrete control cylinders were made and cured in a similar fashion. Three concrete strengths were used, 2000, 3500 and 5000 psi.

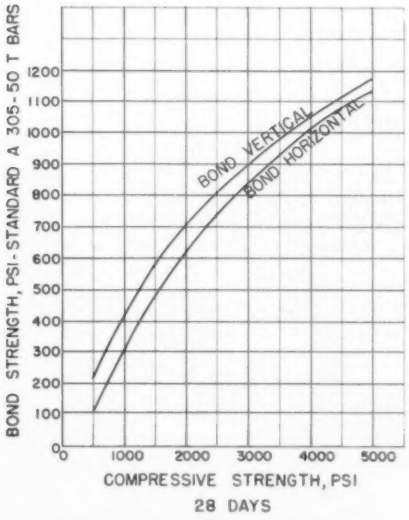


Fig. 9—Bond strength in relation to compressive strength

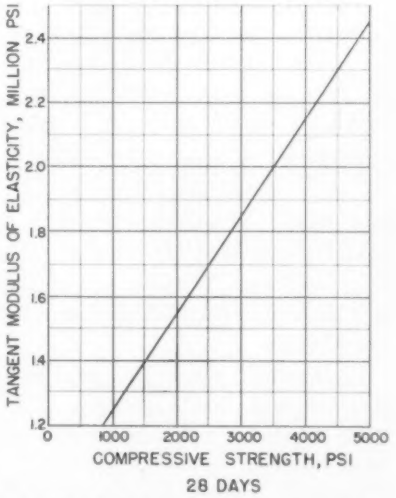


Fig. 10—Relation of tangent modulus of elasticity to 28-day compressive strength

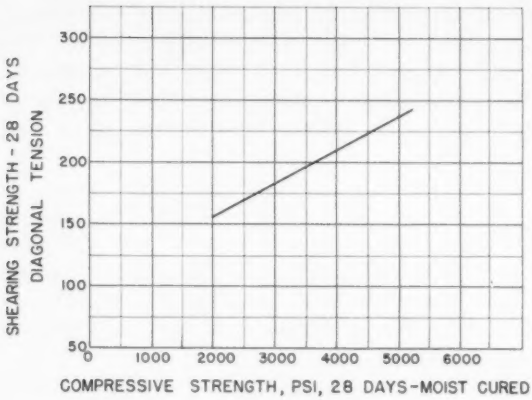


Fig. 11—Shear strength in relation to compressive strength

Deflections at the center of the beam under a concentrated load at the midpoint were measured by a dial gage on one side of the beam. The dial gage was supported on an aluminum bar with ends attached to the sides of the beam over the supports. The gage rested against angle clips attached to the concrete by screws.

The relationship between the shear strength and the compressive strength of the concrete is shown in Fig. 11.

Thermal conductivity

Tests for K factor were conducted on 1 in. thick specimens in a guarded hot plate apparatus according to ASTM C 177-45 requirements.

Thermal conductivity, Btu per sq ft, per hr, per deg F per in., is plotted against compressive strength in Fig. 12. Values varied from 2.40 at 1000 psi to 2.65 at 3000 psi, and up to 3.35 at 5000 psi.

FIELD CONTROL

Problems of field control of batching plant operations and of carrying into the field the lessons which have been learned in the laboratory have been adequately covered by Murlin and Willson* in a paper with which most concrete engineers are familiar. Problems with this local aggregate were the same and most of them solved themselves after a little experience with the new material and after personnel at the batching plants became familiar with handling it.

In the field aggregates were received as coarse aggregate and a blend of medium and fine aggregate. In the dry state, considerable segregation occurred; there was considerable loss of fines from the conveyor system and an attempt to batch dry aggregate was not recommended since the laboratory experience indicated that pre-wetting was desirable. Therefore a spray system was installed on the belt conveyors, uniformly introducing moisture

*Murlin, John A., and Willson, Cedric, "Field Practice in Lightweight Concrete." ACI JOURNAL, Sept. 1952, *Proc.* V. 49, pp. 21-36.

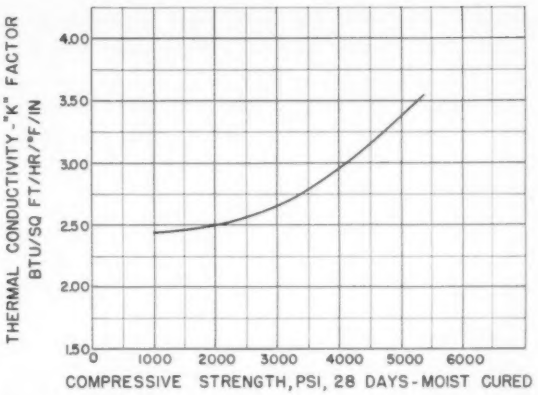


Fig. 12—Thermal conductivity in relation to compressive strength

into the material before stockpiling. The aggregates were only partly saturated and segregation was reduced to a minimum; there was no loss of fines and the material moved easily. All material going into the weigh bins came only through stockpiles which had been treated in this manner and which had had time to stabilize their moisture content. No unloading from cars directly into the weigh bins was practiced. This apparently was the only way to obtain a steady flow of constantly moist material.

Determinations for moisture content are made by drying samples of the aggregates completely on a hot plate. This takes time, and again if the supply of material is not constant in water content, the determination is completely worthless. In a central mixing plant this problem can, however, be quickly solved when the concrete as mixed can be seen and slump tests and weight tests run immediately so that proper adjustments can be made.

Moisture corrections for the wet weight of aggregate to be used are based on the moisture-free material so that a given volume of dry aggregates enters the batch. Water in the aggregate is subtracted from the total water required by the mix proportions and that amount of mixing water added. Experience has shown that if the exact mixing cycle as outlined is used the batch will usually be as desired, or at least as close as is considered good practice in heavyweight concrete. A minimum of trouble is encountered here because the aggregate which is being used absorbs most of the water which it will absorb very rapidly at first, and then absorbs only slowly after this point is reached. Whether this would hold true for many other aggregates is a point in question. If minor changes in water content are needed, they are made immediately on the assumption that the moisture content of aggregate or the amount of water absorbed has changed, and the end product is adjusted accordingly until additional checks can be made.

With regard to the mixing cycle, the following routine is used at a central mixing plant which regularly produces lightweight structural concrete. About 90 percent of the mixing water is added first along with the air entraining agent. Actually the air entraining agent is introduced into the water line

before it reaches the mixer and dispersal is speeded up by this operation. Aggregate is added and allowed to absorb for 1 min. The cement is then added and the entire batch allowed to mix an additional minute. The total time is, therefore, a little longer than customary in other regions, but so far no problems have been encountered in maintaining production volumes. If such a problem arises, some thought will be given to reducing this cycle somewhat, if possible, or increasing mixer capacity.

There has been little difficulty in loss of slump from plant to job. The practice of using a pre-wetted, partially saturated aggregate, then allowing it to mix with the water and air-entraining agent for 1 min, then adding cement, has indicated that the aggregate does absorb the same amount of water from batch to batch during the day, and that no drying out occurs after mixing. The practice of speeding up mixer trucks just before discharging has been followed in this area with the same success as elsewhere.

The length of time to determine the air content of fresh concrete by the rolling meter method is impractical during fast operations at the plant. Therefore, for routine checking, the unit weight of the concrete is used as a check on the air content. A certain weight is set for a given mix and the amount of air-entraining agent is adjusted periodically to obtain that weight. The actual air content is then checked at infrequent intervals or if a change of yield occurs.

Testing procedures are fairly well standardized and no departure from usual practice is made. Test cylinders are cured at 70 F under moist conditions similar to the practice for heavyweight concrete. The cement factors used are about the same as those used for heavy weight concrete containing the same size aggregate. The success of control also has been comparable to past experience with heavyweight concrete.

CONCLUSIONS

1. A definite need has existed for more reliable methods of structural lightweight concrete design and for improvement in the methods of achieving uniformity in field batching control.

2. Comprehensive laboratory investigation has been essential not only to develop such new methods of proportioning and control, but also to evaluate the performance of the lightweight concrete prepared under such methods.

3. The use of the crushing test for determination of the strength of lightweight aggregate does not bear a reliable relationship to the strength potential of such aggregate in concrete. A compaction test may, however, offer possibilities of an indication of the proper proportioning of fine and coarse aggregates.

4. Higher ratio of fine aggregate to total aggregate than is the case with heavyweight sand and gravel is essential to proper workability in lightweight concrete proportioning. From 50 to 65 percent by volume of total aggregate should pass the No. 4 sieve, depending upon cement factor desired. Percentage

of fine aggregate (expressed as percent of total aggregate) should decrease approximately 2 percent for each 1 sack increase in cement content.

5. Absolute minimum percentage of fine aggregate is point where harshness begins to develop at the selected entrained air content. Actual deficiency of fine aggregate is to be avoided in structural strength ranges.

6. The use of entrained air in lightweight concrete is desirable for workability and reduction of water content. Entrained air must be limited to that amount which provides desired workability. Excessive amounts of entrained air will reduce concrete strength at any given cement factor.

7. Entrained air content for workability should range from 8 percent by volume at 3.0 sacks of cement per cu yd to 1 percent by volume for 9.0 sacks of cement per cu yd of concrete.

8. Investigation and use of the usual methods of batching control shows variations in yield and performance beyond that considered acceptable in heavyweight concrete practice. Proposed new mix proportioning and control method incorporating the use of a *specific gravity factor* as determined from prepared concrete provides easy and accurate control of lightweight concrete quality and yield as confirmed both by laboratory investigations and by commercial field batching operations.

9. Performance data on lightweight concretes from laboratory investigations show excellent agreement with results of other investigators. The results determined and reported show that structural quality lightweight aggregate can be relied upon as to performance in concrete, and can effect substantial economies through reduction in dead weight, improved insulation, and other qualities.

10. Results in this investigation were obtained under conditions of strict laboratory control and curing. Expected variations of materials or conditions in the field may necessitate somewhat higher cement factors in actual field practice.

Discussion of this paper should reach ACI headquarters in triplicate by Apr. 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

The widening of the Bordeaux Bridge (L'élargissement du pont de Bordeaux)

R. BENQUET, *Annales des Ponts et Chaussées* (Paris), V. 126, No. 6, Nov.-Dec. 1956, pp. 755-778
Reviewed by ARON L. MIRSKY

The only crossing of the Garonne in a distance of 125 km, the original bridge, a multiple-arch masonry structure constructed during the first quarter of the 19th century, was only 15 m wide between parapets; the roadway was only 7.5 m wide, and there were two 2.5-m sidewalks and two 1.25-m step-curbs. In 1839 the space between parapets was "modernized" to provide a 10.4-m roadway and two 2.3-m sidewalks. And thus it remained until 1953, when it was further modernized. This time the existing parapets, sidewalks, and spandrels were removed and reinforced concrete extensions constructed to give a 19-m width center to center of railings and a 12-m roadway, two 1.5-m sidewalks and two 1.5-m cycle paths. Concrete work, railing, and lamp standards were carefully designed so that the reconstructed bridge retains the flavor of the original.

Progress of the Clifton Bridge, Nottingham

The Engineer (London), V. 203, No. 5289, June 7, 1957, pp. 872-873
Reviewed by ARON L. MIRSKY

Brief description of interesting three-span prestressed concrete bridge over the River Trent. Main span, 275 ft long, consists of a 100 ft suspended span and two cantilever arms each 87.5 ft long. Each 125-ft end span is a "balancing" cantilever, whose outer end is attached to a "flexible" column which is designed to resist temperature movement of the bridge flexibility and thus eliminate undesired variations in cable stresses.

The cantilevers were built by the "free-cantilever" method, using a gantry or traveling platform provided with hydraulic jacks to take up the deflections due to the concrete being placed, so that the green concrete remains unstressed and on correct line. The suspended span consists of seven T-beams, cast on an already-completed portion of the bridge, partially prestressed, and placed in position with the aid of Bailey bridging. The same method was used for the viaduct spans at one end of the structure, as well.

Prestressed girder bridges built without scaffolding (Lehrgerüstlose vorgespannte Balkenbrücken)

HERMANN BAY, *Der Bauingenieur* (Berlin) V. 31, No. 11, Nov. 1956, pp. 412-420
Reviewed by ARON L. MIRSKY

Descriptions of some nine large bridges of prestressed concrete, constructed of individual members (beams) or entire sections cast near the final location, moved into place, and tied into a monolithic whole. The methods used are varied and ingenious.

The Hindan Bridge (in Danish)

TORBEN SKIELLER, *Ingeniøren* (Copenhagen), B Edition, V. 66, No. 25, Sept. 15, 1957, pp. 653-662
Reviewed by JESPER STRANDGAARD

Describes the design and construction of a new prestressed concrete superstructure for a highway bridge near New Delhi, India. Using the existing piers, the new superstructure consists of five simply supported spans approximately 90 ft each, each span consisting of three precast, prestressed (Freyssinet

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 7, Jan. 1958, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

system), T-shaped concrete beams spaced at 10 ft 6 in., with a cast-in-place slab between flanges. Roadway width is 24 ft. The deck slab was designed in accordance with the Guyon method, and the distribution of the vehicular load between the girders was computed in accordance with Courbon's method. The deflections measured under a realistic test loading verified the assumed design procedure.

Post-war developments in German steel bridges and structures

G. Bernard Godfrey, *The Structural Engineer* (London), V. 35, No. 2, February 1957, pp. 53-68
Reviewed by C. P. Siess

One portion is devoted to bridges with composite concrete decks. The new German Standard for composite construction, DIN 1078 (1954), is reviewed briefly.

Construction

A parking garage in Copenhagen (in Danish)

Johs. Ingwersen, *Ingeniøren* (Copenhagen), B Edition, V. 66, No. 24, Sept. 1, 1957, pp. 645-650
Reviewed by Jesper Strandgaard

Describes the design and construction of a 180 x 90 ft parking garage with seven decks and space for 365 cars. No ramps or elevators are used, but all decks are sloped 5 percent, in opposite directions for each half of the building, with a cross-over between decks in the middle of the building. Cars are parked perpendicular to slope. Conventional beam and slab reinforced concrete construction with precast façade elements.

Orebro water tower

The Engineer (London), V. 204, No. 5298, Aug. 9, 1957, pp. 208-209
Reviewed by Aron L. Mirsky

All-too-brief description of very interesting Swedish water tower, characterized by pleasing appearance, economical and up-to-date design, and low cost. Structure, mushroom-like in apearance, has a capacity of 2 million imperial gallons; top water level is 160 ft above the ground, depth of water in the storage tank is 44 ft; maximum diameter is 146 ft. The conical roof, which will house a restaurant, is surmounted by a 100-ft television and radio transmission tower. Tank was constructed, prestressed, and tested on the ground and then jacked to final vertical elevation; provision is made for jacking it still higher in the future, should more head be needed.

Architectural records (Documents d'architecture)

Cahiers, Centre Scientifique et Technique du Bâtiment (Paris), No. 27-233, 1956, 24 pp.
Reviewed by Henri Perrin

Describes four buildings (a school, a bus terminal, a market hall, and an office building) recently built in France. General features and techniques used for construction are briefly stated. Plans, sections, and details accompany photographs taken at various erection stages.

Construction Techniques

Experiment in the use of "cold concrete" (in Russian)

B. D. Kovner, *Beton i Zhelezobeton* (Moscow), No. 2, Feb. 1956, pp. 47-49
Reviewed by K. P. Karpoff

A general discussion of a method for preparation of a concrete mix at a temperature below the freezing point (max −22 C). This method is based on the addition of different concentrations of $CaCl_2$ and NaCl solutions to the mix, depending upon the minimum temperature of the air. Preparation of the "cold concrete" mix does not require preheating aggregate; consequently, the cost of concrete per unit volume only increases 2.7 percent in comparison with the standard costs under normal meteorological conditions.

Technical records of new materials and construction processes (Agrément technique des matériaux nouveaux et des procedes non traditionnels de construction)

Cahiers, Centre Scientifique et Technique du Bâtiment (Paris), No. 26-227, 1956, 60 pp., and No. 27-234, 1956, 34 pp.
Reviewed by Henri Perrin

Presents specifications for various processes used in France for mass-produced buildings. Precasting methods, type of materials, quantities, labor, etc. are described. Some typical details are also given.

Cahier No. 26-227 includes seven papers dealing with walls, six with masonry, and one with prefabricated houses.

Cahier 27-234 includes one paper for each of the following items: walls, masonry, slab partition wall, electrical installation, and foamed concrete, as well as three papers on roofing. Although designed for European standards, some of them may suit American conditions.

Steam curing of concrete (Dampfhärtung des Betons)

O. Graf, *Forschritte und Forschungen im Bauwesen* (Stuttgart), Series C, No. 3, 1954, 24 pp.
Reviewed by Patrick E. Murphy

An extensive series of tests under the supervision of the late Professor Graf examine those variables which affect the flexural and compressive strength of concrete up to 28 days when steam cured by either the low-pressure or high-pressure process. These included conditions and time of storage before and after steam curing, different cements and aggregates in varying proportions, duration of steam curing, and temperature and pressure during steam curing. The results obtained using different cements indicate that this is definitely an important factor. The excellent performance of blast-furnace slag cements (*hochofen*) compared with portland cements is to be noted. Also of interest are a few tests on concrete cured in hot water at 50 C to 70 C which indicate that such curing is more effective than low-pressure steam curing and less detrimental to the 28-day strengths.

Design

Analytical and graphical methods for determination of maximum boundary stresses in eccentrically loaded elements of circular hollow cross section without consideration of the tension zone (in Serbian)

Globocnik St., *Naše Gradevinarstvo* (Belgrade), V. 11, No. 7, July 1957, pp. 189-193
Reviewed by J. J. Polivka

Simplified structural analysis of hollow circular elements built in materials of negligible tensile strength, e.g. chimneys in plain concrete, concrete blocks; gives tabulated values for variable dimensions and eccentricity. The method is applied to an example and the results are compared with those published in Emperger's *Handbuch für Eisenbeton*.

General chord-formula (Die allgemeine Sehnenformel)

G. Worch, *Der Bauingenieur* (Berlin), V. 31, No. 10, Oct. 1956, pp. 373-374
Reviewed by Aron L. Mirsky

Derives expression for change in span of curved beam treated as series of short chords pinned at their joints and loaded at the joints only.

Artificial weight and its applications in reinforced-concrete construction (La pesanteur artificielle et ses applications dans la construction en béton armé)

Roger Takacs, *L'Ingenieur* (Montreal), V. 43, No. 170, Summer 1957, pp. 25-29
Reviewed by Aron L. Mirsky

Study of a method of increasing the stability of various structures subjected to horizontal forces, by increasing the "apparent weight." This is accomplished by means of prestressing, using bedrock or a mass of earth (via friction on the column of concrete encasing the tendons: i.e., a friction pile in reverse).

It is to be noted that this concept has already been applied in the case of dams, notably by Andre Coyne.

Contribution to the problem of the horizontal wall pressure during the emptying of slender silos (Ein Beitrag zur Frage der horizontalen Wanddrücke bei der Leerung von engräumigen Silos)

A. Mehmel, *Der Bauingenieur* (Berlin), V. 31, No. 10, Oct. 1956, pp. 377-379
Reviewed by Aron L. Mirsky

Analysis of pressures (including dynamic factors) on a coal silo, steel-framed with reinforced concrete walls, which exhibited signs of distress. Author comments that further investigations, on models and full-sized structures, appear urgently needed.

An aid in the preliminary design of prestressed concrete beams

Jack R. Clanton, *The Trend in Engineering at the University of Washington*, V. 9, No. 2, Apr. 1957, pp. 28-31
Reviewed by Aron L. Mirsky

To facilitate preliminary design and checking of prestressed I-shaped beams, author presents charts giving section modulus (about centroidal axis) and area versus flange width,

for several combinations of over-all depth, web thickness, and flange thickness. Flange and web proportions in general follow the recommendations of Y. Guyon (*Prestressed Concrete*, New York, Wiley, 1953), with 3 x 3-in. fillets used throughout. Governing conditions assumed are zero tension at top and full allowable compressive stress at bottom of beam, at time of prestressing, with beam weight acting.

Design and calculation of north-light shells in prestressed concrete for 2 × 131 ft continuous span (in Dutch)

A. M. Haas, *De Ingenieur* (Utrecht), V. 68, No. 15, Apr. 1956, pp. 35-46

Applied Mechanics Reviews Sept. 1957

Originally, span of 2 × 164 ft had been specified of which three preliminary designs are shown: two in shell construction and one of the windowless type (also in prestressed concrete).

For the span of 2 × 131 ft which has been chosen and which is now under construction, a simplified method of calculation developed by Mr. van der Eb was followed. The advantage of using prestressed concrete is explained, especially when curved cables are used. For this span the beam method of Professor Lundgren led to the large deviations in the M_ϕ moments.

For a few characteristic cross sections, the values of the normal and shear stresses are shown.

An investigation into the stress distribution in pile caps with some notes on design

Noel Bambury Hobbs and Philip Stein, *Proceedings*, Institution of Civil Engineers (London), V. 7 (session 1956-57), July 1957, pp. 599-628

Reviewed by Aron L. Mirsky

Author develops a stress function applicable to any rectangular section (block, wall, or slender beam) loaded in any desired manner on opposite faces. Solution using this stress function is rigorous and fulfills all boundary conditions.

Results for two-pile caps lead to a simple expression for the tensile force on a central vertical plane. They also show that if the reinforcing steel is bent upwards slightly, it serves also against diagonal tension. Experimental data from tests of some 70 one-third scale models are adduced to confirm analytical results; they also indicate that if reinforcing steel is provided with a rigid welded anchorage, it may be reduced in amount for added economy. Author indicates these results should be applicable to multi-pile caps as well.

On the nonlinearity of proving rings

W. H. Wittrick, *Australian Journal of Applied Science* (Sydney), V. 8, No. 3, Sept. 1957, pp. 145-150

Develops theory showing that load on a proving ring is not proportional to deflection. Corrections in practice may vary by ± 4 percent over the operating range. The stiffness is not a constant but rather may be approximated as a linear function of deflection. Fair agreement with experimental results was achieved.

Equations for improving the precision in determining the intrinsic stresses in concrete and its reinforcement (in Russian)

S. A. Dmitriev, *Beton i Zhelezobeton* (Moscow), No. 2, Feb. 1956, pp. 53-58

Reviewed by K. P. Karpoff

The article presents a brief derivation of the mathematical equations for determining value of intrinsic stress in the reinforced concrete elements. The mathematical study was made for an elementary section with single and double reinforcement. The author analyzed each elementary section using a derivation of the intrinsic stress equation for: (1) Elementary section prior to a binding effect of concrete at the reinforcement, (2) the elementary section just after binding of concrete around the reinforcement but prior to settlement and creeping of the concrete, (3) elementary section after settlement and creeping of the concrete as a basis for the final equations derived for the design of reinforced concrete structure.

In conclusion, the author made the following statements:

(1) Derived equations guarantee a more precise determination of the strain condition and the value of intrinsic stresses in elementary reinforced concrete section, which may result in formation of undesirable cracks in the concrete.

(2) By use of the equations in the design of reinforced concrete structures, it is possible to determine the sections in the structure

which are affected and also which are not affected by the intrinsic stresses. Consequently, the elements of the structure affected by these stresses can be redesigned permitting the desired limiting value of the intrinsic stresses.

(3) Equations, as they are expressed in the article, present realistic values for use in designing reinforced concrete structures, particularly where high precision in determining stresses is necessary. However, these equations can be simplified when greater precision in computation of the stresses is not required.

Ultimate strength of slender prestressed concrete columns

PAUL ZUNG-TEH ZIA, *Engineering Progress at the University of Florida*, V. 11, No. 7, July 1957, 16 pp. (Technical Paper Series, No. 131)

A theoretical application of the ultimate load theory of reinforced concrete to predict the ultimate strength of slender columns in prestressed concrete. Includes comparisons with available data and recommended limits of prestressing for practical applications.

Structural air-raid protection of tall buildings (Baulicher Luftschutz von Hochbauten)

HANS MINETTI, *Der Bauingenieur* (Berlin), V. 32, No. 7, July 1957, pp. 239-244
Reviewed by ARON L. MIRSKY

Background material for, brief commentary on, and justification of the "Specifications for Structural Air-Raid Protection of Tall Buildings—Specifications for Design and Execution" of the Federal Housing Ministry and the Federal Ministry of the Interior, of Germany. Author saves his clincher—photographs of several Hiroshima buildings after the atom bombing—to the last, after he has shown that, for example, compliance with the new specifications adds on the average only about 5 percent to the cost of construction.

In view of the as-yet-in-large-part-unremedied paucity of information on the atom and hydrogen bombs, their effects, and methods of protection against them, and in further view of the undoubted martial proclivities of the Europeans, Americans may well be interested in studying this paper in conjunction with, say, that by C. S. Whitney, B. G. Anderson, and E. Cohen (ACI JOURNAL, Mar. 1955, *Proc.* V. 51, pp. 589-683).

Dimensioning of vibrating foundations (in Polish)

IGOR KISIEL, *Archiwum Inzynierii Ladowej* (Warsaw), V. 1, No. 1-2, 1955, pp. 85-123
Reviewed by STEVEN GALEZEWSKI

Simplified method for direct dimensioning of foundation blocks supporting machinery is proposed. It is shown that for a relatively low frequency of exciting forces, a high weight is not necessary to insure a proper functioning of the foundation. Good graphs and tables are given to speed computations.

Design for minimum weight of simply-supported flat grillages to withstand a single concentrated load

J. CLARKSON, *Transactions*, North East Coast Institution of Engineers and Shipbuilders (Newcastle-upon-Tyne), V. 73, No. 1, Jan. 1957, pp. 145-155
APPLIED MECHANICS REVIEWS July 1957 (Grover)

Elastic design of flat orthogonal grillage or gratings (for such uses as bridge decks, floors of buildings, ship-bottom structures, etc.) is considered from the viewpoint of minimum weight to withstand concentrated loads. Calculated values of bending moments are provided for a considerable range of "uniform" grillages (equal and evenly spaced beams in each direction). A suggested procedure for design use is illustrated by a numerical example.

Vibration of frames

R. E. D. BISHOP, *Proceedings*, Institution of Mechanical Engineers (London), V. 170, No. 29, 1956, pp. 955-967; discussion, p. 968
Reviewed by ARON L. MIRSKY

A sequel to author's previous paper (*Proceedings*, V. 169, No. 51, 1955, pp. 1031-1050; "Current Reviews," ACI JOURNAL, July 1957, p. 78), paper presents method of calculating the natural frequencies of plane frames composed of prismatic members.* Method is numerical and based on tables in previous paper;* it is not restricted to the fundamental, and permits checking a given frame for possible resonance in a given frequency range without first having to calculate all lower natural frequencies. Experimental verification of method is included.

*However, author notes that a method for calculating the receptances of beams of nonuniform section is available (*The Engineer*, V. 202, 1956; Dec. 14, pp. 838-840; Dec. 21, pp. 874-875) and that method of current paper is thus not restricted to frames composed of prismatic members.

Materials

Preventing corrosion of steel reinforcements in concrete

C. COPPARD, *Prefabrication* (London), V. 4, No. 47, Sept. 1957, pp. 506-507

Describes a newly developed process for coating steel reinforcement to prevent its corrosion in reinforced concrete. The process consists of three stages: (1) mill scale removal, which may be chemical or mechanical; (2) "passivating" the steel, which consists of a prime coat of an insoluble phosphate; (3) a final sealer, which is claimed to be unaffected by moisture, acid, or alkali, and to offer a strong bond between metal and concrete.

Grout for prestressed concrete (Einpressmörtel für Spannbeton)

K. WALZ, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 2, Feb. 1957, pp. 53-56

AUTHOR'S SUMMARY

Essential properties that grout should possess have been ascertained by investigations, and appropriate testing methods have been developed. The problem that future work in this field will have to solve is the development of grouts which, while possessing adequate fluidity, will undergo only a small amount of shrinkage and not be liable to water segregation.

Research on high turbulence mixing of cement suspensions (Recherches sur le malaxage a haute turbulence des suspensions de ciment)

MICHEL PAPADAKIS, *Revue des Materiaux* (Paris), No. 498, Mar. 1957, pp. 69-78

Reviewed by PHILLIP L. MELVILLE

A laboratory study of grouts produced in a "high turbulence" mixer. The mixer is described. Viscosity, flow, stability indicate that the resulting grout is well suited for injection work. Four references given.

Barium and strontium cements (Barium- und Strontium-Zemente)

A. BRANISKI, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 5, May 1957, pp. 176-183

Reviewed by H. H. WERNER

The work of Eskola and Greig on strontium and barium silicates is investigated; eight special cements with barium and strontium oxides were prepared and tested. Valuable properties of some of these cements are: exceptionally high degree of resistance to attack by sea water, exceptionally good refractory properties, and excellent protection against x-rays and gamma rays.

Epoxide resin based finishes for the protection of alkaline building materials

H. R. TOUCHIN, *Corrosion Prevention and Control* (London), V. 4, No. 3, Mar. 1957, pp. 70-72

Reviewed by WILLIAM R. LORMAN

The ideal coating for mortar, concrete, stucco, brick, or plaster surfaces subject to corrosive influences should possess the following characteristics: (1) good adhesion, (2) elimination of efflorescence, (3) continuity, (4) protection against corrosive influences, and (5) long life. Epoxide resins crosslinked by amines or polyamide resins fulfill these requirements. Finishes are applied by brush or spray; although hard dry within 12 hr, ultimate chemical resistance is attained 7 days after application. Satisfactory finish is obtained usually with two coats; the initial coat covers about 25 sq yd per gal. Cost is comparatively high. These finishes insured reduced fungus or mould growth. Concrete surfaces painted with epoxide resin based finishes successfully withstand deteriorative effects common to food industries, chemical works, plating shops, and pickling plants.

Experimental use of the vibro-ground cement for manufacturing precast reinforced concrete (in Russian)

A. A. YUKOUSHEV and G. S. PLOTKE, *Beton i Zhelezobeton* (Moscow), No. 2, Feb. 1956, pp. 41-45

Reviewed by K. P. KARPOFF

The commercial cement, before being used in concrete mixture, was subjected to grinding in the specially designed vibro-mills. The ground cement has a high value of fineness; consequently, the activity of cement increases from 55 to 60 percent, water-cement ratio increases from 0.33 to 0.345, and the strength of concrete increases from 25.8 to 40 percent in comparison to the concrete made from commercial cement. To support the above statements, the article provides the laboratory test results and also the illustrative photographs of the scheme for installation of the vibro-mills in the precast concrete plants.

Stucco on insulating concrete (in Swedish)

HJALMAR GRANHOLM, *Cement och Betong* (Malmo), V. 30, No. 3, Oct. 1955, pp. 124-158
Reviewed by BENGT F. FRIBERG

Account of failures of stucco exteriors on insulating concrete block walls, showing lack of adhesion between plaster and lightweight concrete base, and extensive cases of water damage. Warning against stiff plaster with high modulus of elasticity of surface compared to base, and against thick stucco surfaces; lasting adhesion of thin stucco much more likely. Examples of successful stucco installations shown. Separate protective surfaces, such as asbestos cement panels, are recommended for insulating concrete walls in difficult climates.

Protection of structural concrete

P. B. CORMAC, *Corrosion Prevention and Control* (London), V. 4, No. 3, Mar. 1957, pp. 82
Reviewed by WILLIAM R. LORMAN

Concrete surface coatings that incorporate epoxide resins theoretically offer excellent adhesion and film continuity; but in actuality this is not so because concrete surfaces normally contain minute irregularities caused by entrapped air, traces of form oil, and moisture from within the mass. The problem can be solved by superimposing the epoxy resin finish coat upon a primer coat which is comprised of a mixture of hydraulic cement and natural rubber latex. The two coats become integrated chemically and are inseparable. This system is used successfully in Switzerland and England for protecting concrete storage tanks used in the wine and food industries. Such latex epoxide resin coatings have been employed also in repairing concrete beams and columns of an acid concentration unit where spillage and fumes had attacked the concrete sufficiently to cause some exposure of steel reinforcement.

Corrosion-resistant floors

V. EVANS, *Corrosion Technology* (London), V. 4: No. 3, Mar. 1957, pp. 95-97; and No. 4, Apr. 1957, pp. 120-124
Reviewed by WILLIAM R. LORMAN

Part 1 of this treatise is concerned with proper planning of concrete subfloors, interposed impervious membranes, and corrosion-proof finishes. Discusses characteristics of materials employed for impervious interlayers. Part 2 offers technical details relative to hydraulic cements, latex cements, cement mortars, and asphaltic compounds. Floor finishes are divided into two groups, namely, monolithic and brick or tile. Contains specific recommendations concerning protective applications for concrete floors in chemical industries, descaling or pickling plants, plating shops, breweries, dairies, sugar factories, and other food industries. Sixteen references are cited with regard to the technology of cementitious materials adaptable to making concrete floors corrosion-proof.

Pavements

Measurement of strains in a concrete road slab subjected to heavy indivisible loads

Research Note RN/3050/JJT, Road Research Laboratory, Department of Scientific and Industrial Research, Harmondsworth, England
SUMMARIES OF ROAD RESEARCH NOTES No. 57, Sept. 1957

To assess the capabilities of concrete road slabs for carrying heavy indivisible loads, strain measurements were made while various types of vehicle traveled at creep speeds across a slab 30 ft long, 12 ft wide, 9 in. thick.

In general, with the slab thickness and quality of subgrade studied, the very heavy loads produced strains which were no greater than those caused by normal loads except when applied near the undoweled joints at the ends of the slab.

Precast Concrete

Construction of the precast concrete sections in the open air during the winter (in Russian)

E. K. GRABINSKY, *Beton i Zhelezobeton* (Moscow), No. 2, Feb. 1956, pp. 45-47
Reviewed by K. P. KARPOFF

Describes preparation of the concrete mix with addition of sodium chloride (NaCl) and calcium chloride ($CaCl_2$) solutions, placement of the concrete in the forms by use of a preloaded (30-35 g per cm) vibration method, and the curing method of concrete in the dry air chamber for 12 hr with gradual rise in temperature to 50 to 60 C. The statement was made that this method increases the strength of concrete 23 percent above that

for the standard, and also shows the decrease in cost in the construction of precast concrete parts. In general, discussion of the concrete temperature control includes the laboratory test results and evaluation of the dry air curing method.

Exposed aggregate surfaces for architectural concrete panels (in Swedish)

S. LINDBERG, *Cement och Betong* (Malmo), V. 30, No. 4, Dec. 1955, pp. 222-235
Reviewed by BENGT F. FRIBERG

Manufacturing procedures and appearance of permanent exposed aggregate surfaces, including recommended mix, and manipulation for various treatments, water jet, rubbing and brushing, aggregate bedding in sand, surface retarders, acid washes, and rubber formed surfaces.

Prestressed Concrete

Some tests on the effect of time on transmission length in pretensioned concrete

G. D. BASE, *Magazine of Concrete Research* (London), V. 9, No. 26, Aug. 1957, p. 73-82

A certain amount of work already published has indicated that the transmission length in pretensioned concrete increases very considerably with time.

Recent work by the Cement and Concrete Association on 0.2 in. diameter smooth wire indicates no such increase in transmission length, there being little movement, over several months, of the point at which maximum concrete strains were reached in the units tested.

Effects of strain theory on prestressing (in German)

N. DIMITROV, *Publication*, International Association Bridge and Structural Engineering (Zurich), V. 16, 1956, pp. 85-100
APPLIED MECHANICS REVIEWS Nov. 1957

In structures in which prestressing is employed, the deformation of the concrete gives rise, according to theory, to additional second-order permanent moments. If the moments due to the prestressing are greater (less) than those resulting from the long-term loadings, an increasing (decreasing) prestressing effect, which depends on time, is obtained. The accurate determination of the creep strain m of the soil is of considerable importance from the point of view of the effect of the plastic sensitivity.

The deformation coefficient ν is taken as a criterion. For purposes of practical application, the effect of the additional stresses can be considerable for $\nu < 25$.

"Premo" pipe—a new type prestressed concrete pressure pipe (in Swedish)

NILS DARRE-NILSEN, *Cement och Betong* (Malmo), V. 30, No. 1, Apr. 1955, pp. 63-70
Reviewed by BENGT F. FRIBERG

Describes manufacture of new type concrete pressure pipe, prestressed longitudinally with 3-wire pretensioned strands, and circumferentially with tension-wrapped wire spaced slightly away from the core. The pipe core is placed under high internal pressure while the cover concrete is impact applied and cured. In this prodecure the cover is effectively bonded to the core and both become effectively prestressed longitudinally and circumferentially. Equipment, manufacturing steps, and installations from plants in Norway and Sweden are shown.

Properties of Concrete

Static and dynamic moduli of elasticity of concretes and mortars. Resistance to compression and to tension (in French)

R. DANTINNE, *Bulletin*, CERES (Liège), V. 8, 1956, pp. 155-170
Reviewed by ARON L. MIRSKY

In a previous paper, author showed that certain major defects in concrete had only minor influence on the velocity of propagation of a shock wave; that the grip of the concrete around a steel reinforcing bar was sufficient to transmit a shock wave from one material to the other, independent of any bond between steel and concrete.

In an effort to study this further, author ran comparison tests on regular concrete and on concrete made with coarse aggregate coated with bitumen, to reduce or destroy bond between stone and cement. Results (averages for one series quoted): the densities of the two concretes were almost identical; the compressive strength of the latter concrete dropped to one-third that of the former, but the wave propagation velocity was re-

duced by only 6 percent and the dynamic modulus of elasticity was thus reduced from 340,000 to 300,000 kg per sq cm.

The production of tension by loading test cubes along a diagonal in compression, and some aspects of extensometers with large base lengths as used in the determination of the static moduli of elasticity in these studies, by the usual compression tests, are also discussed, and test results for concretes and mortars stated.

Studies in cement-aggregate reaction

C. E. S. Davis, *Australian Journal of Applied Science* (Sydney), V. 8, No. 3, Sept. 1957, pp. 222-234

A comparison of the expansions of mortar and concrete. An expansive mortar combined with inert coarse aggregate in concrete will also expand. The expansion in the concrete occurs after a greater delay than in the mortar especially along with low alkali content. The age at which the concrete will begin to expand may be predicted from mortar tests. One conclusion is that opal is compatible only with cement containing less than 0.4 percent free alkali.

Concrete deterioration of a foundation

G. M. Idorn, *Acta Polytechnica* (Copenhagen) No. 221, *Civil Engineering and Building Construction Series*, V. 4, No. 3, 1957, 48 pp., 8 Swedish Kr.; also issued as *Reprint* No. 88, Danish National Institute of Building Research, Copenhagen, 12 Danish Kr.

Describes the investigation of concrete deterioration of a foundation for a silo in the harbor area of Copenhagen. Petrographic examination of broken pieces of the deteriorated concrete and of thin sections prepared from concrete specimens were carried out. Chemical analyses were also carried out on concrete samples, and optical measurements were made on secondarily deposited compounds in the concrete. These examinations were supplemented with chemical analyses of ground water samples from the site.

Initial examinations indicated a marked sulfate attack, but the subsequent detailed investigations showed that aggressive carbon dioxide might probably be the primary agent of disintegration. Extensive formations of calcium sulfoaluminate, calcite, and a presumed calcium aluminate hydrate were found as secondary deposits in the concrete. Detailed descriptions of these substances are given. The practical aspects of the findings are mentioned in the closing remarks, in which also the need for further research is emphasized.

Sawdust concrete serves useful purpose

Constructional Review (Sydney), V. 30, No. 8, Aug. 1957, pp. 31-32

Gives some practical data on properties of sawdust concrete in discussion of its applications as an insulating material or for special purposes.

Significance of free lime and calcium aluminate with regard to the sulfate resistance of concrete (Über die Bedeutung des freien Kalks und des Calciumaluminats im Portlandzement für die Sulfatbeständigkeit von Beton)

F. W. Maier-Grolman, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 6, June 1957, pp. 231-238
Reviewed by H. H. Werner

The author defends his theory and observations against those made by F. W. Locher in the article from same issue of the magazine reviewed in December 1957 "Current Reviews," p. 542.

Quick method for determining cement content of fresh concrete

R. E. Tobin, *Concrete Construction*, V. 2, No. 10, Oct. 1957, pp. 8-11

Describes the Willis-Hime method which requires a centrifuge and a heavy liquid media to separate cement from sand. Also describes a newer variation of this method based on apparent specific gravity of an unseparated mixture of cement and sand, requiring less equipment and more adaptable to field usage. Both methods are said to give determinations of cement content to within ¼ sack per cu yd.

Gas concrete (Gasbeton)

Fortschritte und Forschungen im Bauwesen, Series C, No. 4 (Stuttgart), 1956, 61 pp., 14.50 DM
Reviewed by Aron L. Mirsky

A collection of eight papers on cellular (gas and foamed) concrete. Jürgen Brandt writes (pp. 7-15) on the technical and economic possibilities of gas and foamed concrete in Germany. Otto Graf discusses (pp. 16-26) the development of such concretes in Germany: production, properties, use. Ernst Vocke covers (pp. 27-33) production control

in a Ytong plant at Darmstadt. Hermann Schäffler writes (pp. 34-38) on locked-up stresses in reinforced slabs of steam-cured gas and foamed concrete. He also reports (pp. 39-42) on the creep of such slabs. Hubert Rüsch discusses (pp. 43-49) research on the shearing strength of reinforced beams of steam-cured gas and foamed concrete, including pull-out tests on stirrups in such beams. Walter Berger discusses (pp. 50-57) research on the basic principles governing the ultimate strength of such beams. And Karl Gösele writes (pp. 58-61) on a topic dear to the ears of some of us—the acoustic insulation of houses with walls (including party walls) of gas concrete.

Structural Research

Prestressed concrete framework for the Liverpool University Medical School

E. Shepley, *Reinforced Concrete Review* (London), V. 3, No. 6, 1954, pp. 371-392
Reviewed by William R. Lorman

Describes design and construction of prototype prestressed concrete frame which was tested to destruction as a preliminary to the final structural design of the building. The test frame was a full size replica of a frame section located between contraflexure points corresponding to those at an intermediate floor. Diagrams show general arrangement of flexible straining rods, strain gages, and deflectometers. Describes mix design and fabrication of concrete, stressing procedure, and development of cracks; included are graphical relationships of loads, deflections, and end moments.

Experimental studies on composite structures (in Japanese)

M. Wakabayashi, *Report*, Institute of Industrial Science, University of Tokyo, V. 6, No. 2, Dec. 1956, 63 pp.
Applied Mechanics Reviews
July 1957 (S. Okamoto)

The composite structure tested in the present investigation is a monolithic solid structure of concrete having steel bars placed around a fabricated steel core-section. In Japan, composite structures of this kind are usually applied to the construction of buildings with more than six stories.

The tests include: (1) beams subject to bending; (2) columns subject to combined bending and axial forces; (3) beams subject to shear; (4) columns subject to axial and shearing forces; (5) connections subject to bending; and (6) bond strength between steel and concrete.

General

Loading system for creep studies of concrete

C. H. Best, D. Pirtz, and M. Polivka, *ASTM Bulletin*, No. 224, Sept. 1957, pp. 44-47

To simplify the application of constant loads over long periods of time and to permit the use of higher sustained loads, a hydraulic system was developed. Contained herein are the details of this system along with a description of the loading frames used by the authors for creep studies of concretes employing specimens having diameters of 6, 16, and 30 in.

Design for safety and efficiency

A. Pugsley, *The Structural Engineer* (London), V. 35, No. 1, Jan. 1957, pp. 36-40
Reviewed by C. P. Siess

More or less philosophical discussion of efficiency, as measured by economy of material, and safety as a function of variations in loading and strength characteristics and consequences of failure.

Structural problems connected with palletization

Brian Scruby, *The Structural Engineer* (London), V. 35, No. 3, March 1957, pp. 93-106
Reviewed by C. P. Siess

Site selection, layout, framing, cladding, floor finish, and costs are discussed for warehouses to accommodate fork-lift trucks and palletized loads. All structures had floors on ground. Structural design of floors for lift-truck loads is not discussed.

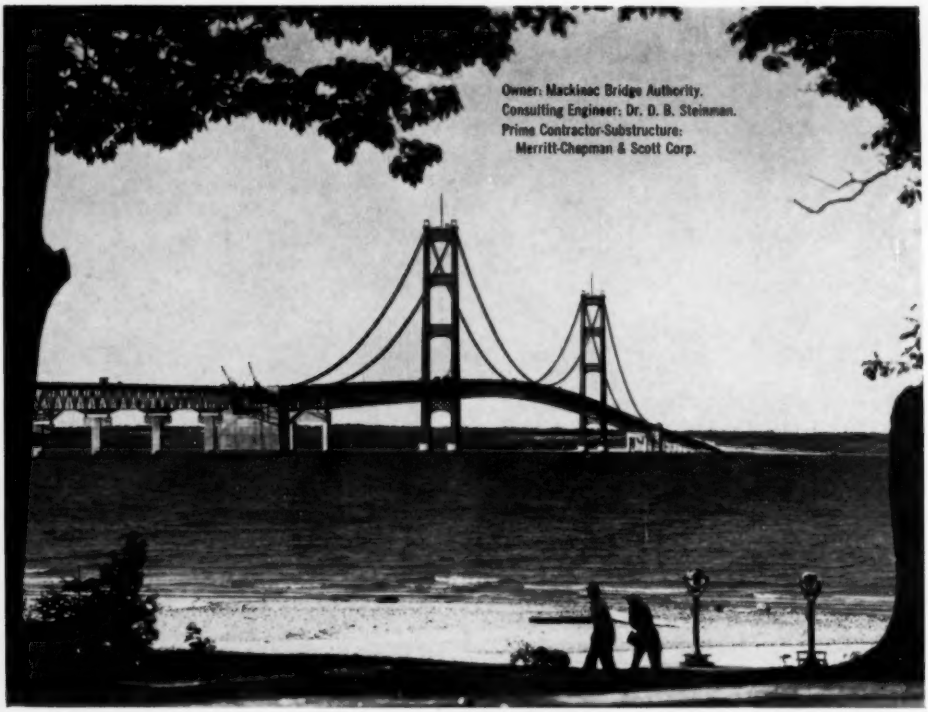
Securing a cliff wall in imminent danger of collapse (Sicherung einer absturzdrohenden Felswand)

E. Fickert and L. Muller, *Der Bauingenieur* (Berlin), V. 31, No. 10, Oct. 1956, pp. 374-377
Reviewed by Aron L. Mirsky

Precipitous cliffs overhanging community of Harburg were made safe with aid of grouted-in anchor bolts and cast-in-place reinforced-concrete distributing beams. An interesting and out-of-the-ordinary job.

ACI News Letter

Vol. 29, No. 7 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1958


Owner: Mackinac Bridge Authority.
Consulting Engineer: Dr. D. B. Steinman.
Prime Contractor-Substructure:
Merritt-Chapman & Scott Corp.

There's more to Mackinac than meets the eye

The graceful superstructure of the new Mackinac Bridge is an outstanding example of bridge building at its finest. Visible for many miles above the northern Michigan countryside, the 552 ft. main towers present a spectacular picture.

A less conspicuous, but equally important part of this engineering triumph, lies in the deep, treacherous waters of five-mile wide Mackinac Straits. Some 451,000 cubic yards of concrete went into the 34 piers comprising the bridge's substructure. One anchorage pier alone contained 91,600 yards, and tower piers were founded as much as 210 ft. below the surface.

The substructure contractor was faced with a formidable combination of hazardous water, unpredictable weather and a tight construction schedule. To minimize delays and risks under such difficult operating conditions, the unique Prepakt® method of concrete placement was selected. How completely this confidence was justified is a matter of record —in fact, a world record.

During the first construction season, the substructure contractor, working with expert Prepakt crews, set new records for underwater concrete placement from a single floating plant. The following year these marks were broken with new world records of 6,250 cubic yards of Prepakt concrete in a single day, 33,068 yards in seven days, and 103,400 yards in a 30-day period. Prominent authorities have stated that without the use of Prepakt concrete, another full year might have been required to complete the Mackinac Bridge. Certainly its use precluded the mobilization of much additional personnel, materiel and floating equipment.

Such performances on tough projects are typical of I-P operations. Patented methods and materials in the hands of experienced personnel offer unmatched speed, economy, consistent quality, and flexibility under adverse conditions. These advantages have been successfully demonstrated on many types of work in every part of the world. For more information on Prepakt concrete and other I-P services, contact: Intrusion-Prepakt, Inc., Room 568-L Union Commerce Bldg., Cleveland 14, Ohio. In Canada: Intrusion-Prepakt, Ltd., 159 Bay Street, Toronto, Ontario.

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ACI

News Letter Contents

January 1958

On the Cover—Striking 20-story cooperative apartment centrally located on Oakdale in Chicago's downtown residential section. Reinforced concrete frame with cantilever floors which serve as sunshades for the glass-walled structure. Milton Schwartz, architect.

—*Photo courtesy Portland Cement Association*

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54th Annual Convention

February 24-27, 1958

Morrison Hotel, Chicago

A 3-day program—packed with action as well as studied contemplation—awaits ACI convention-goers at the 54th Annual Convention next month in Chicago. Following a full day of technical committee meetings and work sessions on Monday, the meeting opens officially on Tuesday morning, February 25, with ACI President Walter H. Price presiding at the first general session.

Complete program beginning on p. 4 includes a short synopsis of each of the technical papers to be presented on the well diversified, highly informative program. Other special events of the annual meeting are listed in the program sequence. The Institute's Turner and Lindau Awards, the Wason Medals, and the ACI Construction Practice Award will be presented at the luncheon scheduled for Wednesday noon, February 26. Full story of the awards and awardees appeared in the December, 1957, News Letter.

The local committee on arrangements, headed by A. Allan Bates, vice-president for research and development, Portland Cement Association, has planned some events of more than usual interest to supplement the technical program and to provide social activities and entertainment for the ladies. Some of these events have been planned by joint action of the ACI committee and the local ASCE convention committee (the American Society of Civil Engineers meets in Chicago concurrently with ACI) in order that registrants and their wives at either of the meetings will have ample opportunity to mingle during both educational and festive occasions.

Special events for ACI ladies include a fashion luncheon plus visits to the popular Don McNeill Breakfast Show and to the world renowned puppet show at the Kungsholm restaurant. Joint activities with ASCE include fellowship hours and a

special joint banquet and dance (not to be confused with the usual convention banquet) featuring an outstanding dinner and a number of nationally noted entertainment acts. Tours of interest will include a visit to the Research and Development Laboratories of the Portland Cement Association in nearby Skokie, Ill. For the banquet and all of the events scheduled for the ladies during the week a book of tickets will be sold at a fraction of the actual cost, thanks to the hospitality of ACI enthusiasts in the Chicago area.

TUESDAY MORNING, FEBRUARY 25

First General Session—W. H. Price, Chairman

Opening Remarks—President Price

Proposed Revision of Institute Bylaws

Proposed Standards:

Committee 208—Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars

Committee 325—Proposed Recommended Practice for Design of Concrete Pavements

Committee 617—Proposed Revision of "Specifications for Concrete Pavements and Bases (ACI 617-51)"

Committee 711—Proposed Revision of "Minimum Standard Requirements for Precast Floor Units (ACI 711-53)"

The New ACI Headquarters Building

Minoru Yamasaki, Yamasaki, Leinweber and Associates, Birmingham, Mich.

Motion picture of ACI building in process of construction

TUESDAY AFTERNOON, FEBRUARY 25

St. Lawrence Seaway—I. L. Tyler, Session Chairman

Sponsored by ACI Committee 207

Features of the St. Lawrence International Power Project—J. Neil Mustard, Hydro-Electric Power Commission of Ontario, Cornwall

Presents the general aspect of the St. Lawrence Power Project bringing out the close international cooperation which was necessary to meet the precise schedule for flooding and power production.

Concrete Technology of the St. Lawrence International Power Project—Harry H. McLean, New York State Department of Public Works, Albany; formerly concrete control engineer with Uhl, Hall, and Rich, Massena, N. Y.

Highlights the differences between Canadian and American practice in all phases of concrete technology. Liberal illustrations will show progress made and methods used on the international structure.

Features of the St. Lawrence International Seaway Project—Gordon Kidd, St. Lawrence Seaway Authority, Montreal

Graphic report of the construction required to build a waterway for heavy ocean-going vessels and the delicately bulky lake transports which travel from the continental coastal waters to the interior. Colored slide illustrations will carry the listener from the Great Lakes, down the St. Lawrence River, through the heavy construction areas, comparing the old with the new. Focuses on the lock navigation facilities, also treats the supplementary remedial construction which requires new bridges, roads, sewers, and water intakes.

Concrete Technology in the St. Lawrence International Seaway Project—M. R. Smith, Corps of Engineers, Washington, D. C.; formerly with the Buffalo District

Briefly describes concrete construction on the United States and Canadian St. Lawrence Seaway, paying particular attention to contrasting practices. Differences include variations in specifications and their effect on the concrete mixtures used and methods of construction.

WEDNESDAY MORNING, FEBRUARY 26

Fatigue of Concrete—Gene M. Nordby, Session Chairman

Sponsored by ACI Committee 215

History of Fatigue of Concrete—Gene M. Nordby, National Science Foundation, Washington, D. C.

Based upon the bibliography now being compiled by ACI Committee 215, Fatigue of Concrete. Paper encompasses the early work of De Joly, starting in 1898, and extends to the present. A brief description of the most important investigations will be given in addition to a summary of their conclusions and implications. The review will cover several phases, such as fatigue of plain concrete in compression, tension, and flexure, and the resultant effects on the elastic properties of concrete. A discussion of the fatigue of conventionally reinforced concrete and prestressed concrete will be included as well as the fatigue of bond specimens. Suggestions will be made as to the direction research should take, and facts of direct use to the practicing engineer will be emphasized as often as the state of knowledge permits.

Effect of Range of Stress and Fatigue Strength of Plain Concrete Beams—J. M. Murdock and Clyde Kesler, University of Illinois, Urbana

Summarizes the results of a series of tests made at the University of Illinois to determine the behavior of plain concrete when subjected to fatigue loading. Places primary emphasis on the probable mode of failure and the effects of the range of stress, and covers the behavior of plain concrete subjected to loads which produce both repeated and reversed stresses. These data are plotted as a modified Goodman diagram for study and to facilitate comparison to the fatigue behavior of metals.

Fatigue Properties of Concrete Beams—Thomas E. Stelson, Carnegie Institute of Technology, Pittsburgh, and John N. Cernica, Youngstown University, Youngstown, Ohio

Eleven concrete beams with regular tensile reinforcement were tested under repeated load applied 320 times per min. The beams were identical except for a slight increase in concrete strength during the period of testing. The ACI elastic design load for the test beams was

2610 lb; ultimate load was 8800 lb. The load causing fatigue failure after 500,000 cycles was 5720 lb. Thus the ultimate load was 3.37 times the design load and the fatigue load was 2.18 times the design load. The test results show greater relative fatigue strength than has been previously reported and show that safety against fatigue failure is greater in beams than in compression members.

Probability of Fatigue Failure of Plain Concrete—John T. McCall, Michigan State University, East Lansing

Fatigue tests were run on concrete beams and the data were analyzed in an attempt to determine the relationship for concrete between stress S, number of cycles to failure N, and probability of failure P. Prismatic concrete beams were loaded in completely reversed bending at the rate of 1800 cycles per min. Beams were tested at two stress levels and the number of cycles necessary to cause failure was determined.

From the data for tests run to ten million cycles, it was concluded that the S-N-P relationship for concrete can be obtained graphically and that one of the mathematical relationships is valid for the concrete tested. It was also concluded that a definite fatigue limit for the concrete studied does not exist (in a range up to ten million cycles) and it was estimated that the probability of concrete exceeding a fatigue limit of ten million cycles is one-half—if the alternating stress amplitude is half of the ultimate strength and the mean stress is zero.

Static and Fatigue Strength of Beams with Tensile Reinforcement—Tien S. Chang and Clyde E. Kesler, University of Illinois, Urbana

The purpose of this paper is twofold: to study the static and fatigue strength in shear and in flexure of concrete beams with tensile reinforcement and to study the transition characteristics of the modes of failures in shear and in flexure under the repeated loading.

Formulas were developed to estimate the static initial diagonal cracking load and the static ultimate moment of beams with tensile reinforcement. Fatigue tests were made during the investigation on simply supported concrete beams of relatively small cross sections (4 x 6 in.) with tensile reinforcement only. Statistical studies of the fatigue behavior with regard to initial diagonal cracking, shear-compression, and fatigue of reinforcement were made.

Construction—Douglas McHenry, Session Chairman

Failures of Concrete Structures—Jacob Feld, Consulting Engineer, New York

Structural failures have occurred at all times in all types of work. Mr. Feld offers a historical survey of concrete failures of the last half century in the United States. It is hoped that publicity of all failures will become more common and both the engineering profession and the construction industry can learn by the mistakes of others and avoid such incidents in their future work.

Control of Concrete Mixes—E. A. Abdun-Nur, consulting engineer, Denver, and J. J. Waddell, Joseph K. Knoerle and Associates, Chicago

Descriptions and data on the application of the basic principles propounded by Messrs. Abdun-Nur and Tuthill at Dallas convention, as applied to a large multicontract highway project, Northern Illinois Toll Highway. A short description of the actual "self-functioning" and automation features that lead to increase in uniformity will be given. Actual data and the statistical treatment and its effect on quality control of concrete mixes when keyed to average strength requirements will be discussed in detail.

Construction Features of the British Columbia Electric Building, Vancouver, B. C.—O. Safir, consulting engineer, Vancouver

Describes the construction of the project's tower. This building puts the normally dead load of structural fireproofing to work in two ways: (1) by changing the block or brick fireproofing around the core to reinforced concrete; and (2) by framing composite beams of steel and concrete out from the core to rest on slim exterior columns. Major advantage of this core (not withstanding the fact that it takes most of the load of the beam and slab off its floors) is the wind-bracing function. Method of analysis used for the core will be covered along with construction techniques.

Wide Span Hyperbolic Paraboloidical Shell—Anton Tedesko, Roberts and Shaefer Co., New York

This shell, 132 x 113 ft, believed to be the widest span hyperbolic paraboloid shell on the continent is now nearing completion in Denver. It is part of Webb & Knapp's new Court House Square Development. Design and construction features will be discussed and illustrated with lantern slides.

WEDNESDAY NOON—Awards Luncheon

WEDNESDAY AFTERNOON, FEBRUARY 26

Structural Design—Phil M. Ferguson, Session Chairman

A joint session with the ASCE Structural Division

Influence of Design and Details on Concrete Deterioration—P. D. Miesenhelder, Association of American Railroads, Chicago

Will deal with deterioration of concrete, which is influenced by or is a consequence of the design of the structure. When deterioration of concrete is observed, it is naturally considered a weakness or shortcoming of the material. But this conclusion is confusing when other portions of the structure in excellent condition were of the same materials and placed at the same time. Examinations of structures which were built in different localities, of different materials, and in different years, reveal the apparent similarity of conditions existing at these points. These conditions directly result from the design of the structure.

Basically the deterioration is the result of freezing and thawing. The difference between affected and unaffected parts is the amount of water absorbed by the concrete. Damaging absorption results from leakage or from poor drainage inherent in the design.

Effect of Axial Force and Shear Strength of Reinforced Concrete—James W. Baldwin, Jr., University of Illinois, Urbana, and Ivan M. Viest, AASHO Road Test, Ottawa, Ill.

An experimental investigation of effect of axial compression on the shear strength of reinforced concrete members without web reinforcement. It involved tests of knee frames with ratios of axial force to shear varying from zero to six, and covering the entire range from failure caused by shear in the absence of axial load to failure caused by eccentric compression. The investigation was an extension of an earlier study involving specimens with the axial load equal to shear and specimens with no axial load. Observed diagonal tension cracking loads were found in good agreement with the results of the earlier study. On the other hand, the shear compression strength was found to increase with axial load considerably faster than

indicated by the earlier tests. A modification of an empirical parameter in an existing theoretical expression for the ultimate strength was found necessary. This modification suggests that the compatibility, as well as the equilibrium of forces is a function of the axial load. The modified analysis was compared with the test data obtained by other investigators.

Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members without Shear Reinforcement—Charles S. Whitney, Ammann and Whitney, New York and Milwaukee

Based on the paper which appeared in the October, 1956, JOURNAL. Embodies radically different method for estimating shear strength of flat slabs, column footings, beams, and

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frames; shear strength is not considered a simple function of concrete strength, but depends largely on the amount of flexural reinforcement and its efficiency. Mr. Whitney's theory is supported by the results of tests covering a wide range of proportions and concrete and steel strengths.

Development of Precast Concrete Girders Reinforced with High Strength Deformed Bars—J. R. Caston and Eivind Hognestad, Portland Cement Association, Chicago

Two 0.38-scale model roof girders were tested to develop an unusual type of precast concrete building frame. The roof girder selected departs from customary practice primarily in its slender and graceful "T" cross section, by its high-strength longitudinal tension reinforcement, and by its inclined stirrup reinforcement. Structural design was based on the ultimate strength design procedure given in the appendix of the 1956 ACI Building Code, with some departures justified by the model girder test results. Twenty 58-ft girders were later manufactured for two laboratory buildings.

Design of Highway Pavements—E. A. Finney, Session Chairman

Sponsored by ACI Committee 325

New Factors in Pavement Design—E. A. Finney, Michigan State Highway Commission, East Lansing

Past work of ACI Committee 325 in relation to concrete pavement design is reviewed. The future work of the committee in the fields of continuously reinforced concrete pavement, prestressed concrete pavement, and concrete overlay is briefly explained. New factors to be considered in concrete pavement design, construction, and performance in relation to the expanded highway construction program are presented, including areas of needed research.

Subcommittee reports as follows:

Prestressed Concrete Pavements

Continuously Reinforced Concrete Pavements

Concrete Overlay for Concrete Pavements

WEDNESDAY EVENING—Convention Banquet

THURSDAY MORNING, FEBRUARY 27

Annual Research Session—George W. Washa, Session Chairman

Sponsored by ACI Committee 115

Brief off-the-record reports of outstanding concrete research now in progress in institutions and industry throughout the nation.

THURSDAY AFTERNOON, FEBRUARY 27

Tour of Portland Cement Association Laboratories

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 214, Fatigue of Concrete

John T. McCall
Michigan State University
East Lansing, Mich.

Thomas Stelson
Carnegie Institute of Technology
Pittsburgh, Pa.

Carl Ekberg
Lehigh University
Bethlehem, Pa.

Committee 322, Design of Unreinforced Concrete

LeRoy T. Oehler
Michigan State Highway Department
Lansing, Mich.

Robert S. Rowe
Duke University
Durham, N. C.

R. C. Sandberg
Architect-Engineer
Rock Island, Ill.

Fritz Kramrisch
Albert Kahn Associates
Detroit, Mich.

A. S. Neiman
U. S. Air Force
Washington, D. C.

Clayton M. Crosier
University of Kansas
Lawrence, Kan.

B. Netupsky
Structural Engineer
Vancouver, B. C.

Frederick P. Weisinger
University of Illinois, Navy Pier Branch
Chicago, Ill.

Committee 323, Prestressed Reinforced Concrete (Joint ACI-ASCE)

Gene M. Nordby
National Science Foundation
Washington, D. C.

Committee 332, Recommended Practice for Residential Concrete Work

John Young
Central Arizona Ready-Mixed Concrete Association
Phoenix, Ariz.

Committee 335, Deflection of Concrete Building Structures

Carlos D. Bullock
Burgwin and Martin
Topeka, Kan.

Charles M. Herd
State of California
Department of Architecture
Sacramento, Calif.

George E. Large
Ohio State University
Columbus, Ohio

Marvin A. Larson
Structural Engineer
San Francisco, Calif.

D. E. Parsons
National Bureau of Standards
Washington, D. C.

Donald R. Peirce
Ammann and Whitney
New York, N. Y.

A. C. Scordelis
University of California
Berkeley, Calif.

Mete Sozen
University of Illinois
Urbana, Ill.

Harry A. Williams
Stanford University
Stanford, Calif.

Committee 622, Formwork for Concrete

Jacob Feld
Consulting Engineer
New York, N. Y.

Committee 714, Concrete Bins and Silos

Paul Rogers
Paul Rogers & Associates
Chicago, Ill.

Keeve Steyn
Structural Engineer
Johannesburg, South Africa

Southworth joins NCSA staff

The National Crushed Stone Association, Washington, has announced the appointment of Bill N. Southworth as assistant to the executive director to fill the vacancy created by the resignation of R. W. Prewitt. Mr. Southworth is a graduate of the University of Southern California.

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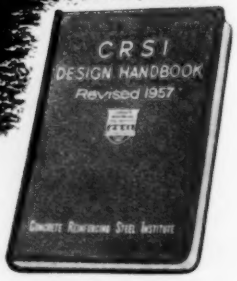

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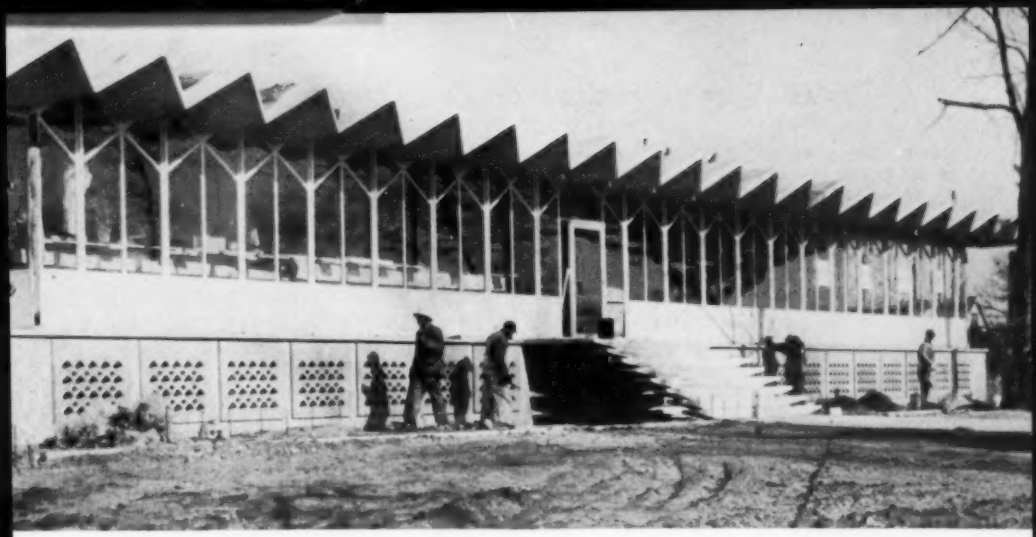


Nearly finished . . .
Front facade of ACI's new headquarters building in Detroit assumes a finished look with glazing completed and interior work proceeding. Here workmen make final check of clean-up on precast grill slabs which shield the building's basement windows

ACI HEADQUARTERS

EXTERIOR AND INTERIOR finishing proceeds at ACI's new headquarters in Detroit, with target date for occupancy set for late January. Glazing, plastering, terrazzo work, precast panels, and masonry installations are part of the "frosting" recently added in and around the striking concrete-roofed structure. Architect for the building is Minoru Yamasaki of Yamasaki, Leinweber and Associates, Birmingham, Mich. Structural design was by Ammann and Whitney, New York and Milwaukee, with Pulte-Strang, Inc., Ferndale, Mich., serving as general contractor.

Precast exposed aggregate panels were set in the aluminum sash to serve as a base for the window walls. These panels, approximately 2 ft square, were made by Michigan Art Marble Co., Dearborn, Mich. The 1⅝-in. slabs were made with quartz aggregate and white portland cement, reinforced with wire mesh. They were cured 72 hr under burlap, and the aggregate was exposed by washing first with water, then with dilute muriatic acid, followed by a third water rinse. Design strength of these Dorix panels is 5000 psi at 28 days.

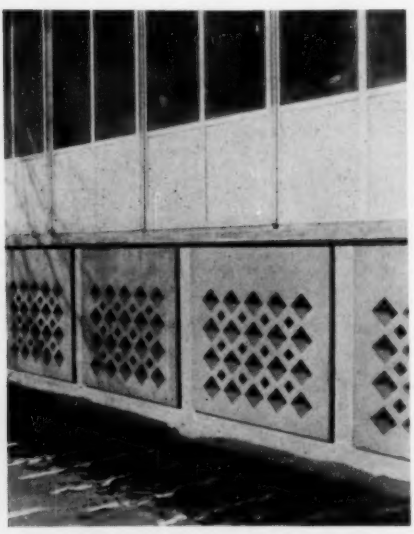
Openwork design in the 3½ in. thick concrete screens in front of basement window openings is part of architect Yamasaki's

Then the wall . . .
Three-course block wall made of slag concrete units will encircle the entire building. Wall now conceals the grill visible in upper photograph

scheme for ornamenting exposed surfaces of the building. Superior Cement Products Co., Detroit, fabricated the specially designed panels as well as precast interior sills for the basement windows and a coping for the masonry wall surrounding the parking lot. The heavy panels were attached to the window frames by bronze dowels at the top, and a galvanized metal anchor plate attached to the cast-in-place exterior window sill was fitted into a recess cast in the base of the concrete slab.

The window grills measure 3 ft 8 in. x 3 ft 10 in. Reinforcement is of ⅜-in. bars, with welded connections. Concrete was a 1:3 mix (by volume) with 1/5 gal. per cu yd of Drycon added for waterproofing; also PH plasticizer was added to improve durability. High-early-strength cement was used along with a 50-50 washed masonry sand as aggregate. The concrete was consolidated in the forms by pneumatic hammer.

Strength tests were made on 2-in. cubes cured 48 hr in their molds before being exposed to laboratory conditions. Average 7-day compressive strength reported was 3079 psi; 28-day test results indicated 3783 psi.



Textural contrast . . .

Light colored exposed aggregate panels were set in sash at base of window wall. Close-up shows smooth gray of pierced concrete grills shielding lower windows

Exterior perimeter wall

Concrete blocks for the "garden" wall surrounding the building were made to architect's specifications by John A. Mercier Brick Co., Dearborn, Mich. Irregularly hexagonal in plan, the blocks were set in three courses. The wall is about 8 ft away from the building on all sides, producing the visual effect of a lengthened base line for the structure.

A specially graded semi-wet slag aggregate was used in producing a mix much wetter than that normally used in block making. Manufacturer's data on the experimentally developed mix indicate that the following amounts of material were used per block: 10 lb cement, 60.5 lb fine aggregate, and 3 lb of "coarse" aggregate. Finished weight per block was 77 lb. Curing was with steam for 24 hr without pressure.

Seven-day strength reported for these masonry units was 1408 psi based on the net loaded area of the block.

REVIEWER NEEDED

Beton i Zhelezobeton

Concrete and Reinforced Concrete is the English title of this monthly magazine in Russian. A volunteer reviewer is needed to keep members informed of pertinent papers therein through the "Current Reviews" section of the JOURNAL. Those interested should write the Secretary of the Technical Activities Committee at ACI headquarters.

Floors and ceilings

Interior finishing work being completed at this writing includes the 2-in. concrete topping for hollow-core units which comprise the main floor.

Interior ceiling line on first floor offices will follow contours of the folded plate

Continued on p. 22

Synopsis of technical papers from ACI regional meeting complete the

Seattle Story

West Coast building projects, research, and developments were spotlighted at the ACI Tenth Regional Meeting, Benjamin Franklin Hotel, Seattle, Nov. 5-6, 1957. Official registration figures totaled 214. Although the majority of registrants came from Washington and Oregon, the meeting also drew members from Canada and from as far east as the Atlantic states.

The program featured three technical sessions and an inspection tour through the Boeing Developmental Center in Seattle, which occupies some 40 acres of land and affords more than 1,000,000 sq ft of floor space.

R. H. Lochow, Portland Cement Association, Seattle, headed local committee

FIRST TECHNICAL SESSION

Moderator for the first technical session on Tuesday, November 5, was H. E. Wessman, Dean of Engineering, University of Washington, Seattle. The session featured research reports from three Pacific Coast schools.

Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings—Jack R. Benjamin and Harry A. Williams, associate professor of structural engineering and professor of civil engineering, respectively, Stanford University, Stanford, Calif.

Scale model one-story reinforced concrete shear walls containing openings were tested at Stanford University as a part of a major study of shear walls sponsored by the Corps of Engineers. The report covered two different investigations.

The first study was concerned with means of analyzing and predicting the behavior of shear walls containing openings of various types. Special steel provided around the openings was kept constant in quantity and was similar to that used in conventional practice. Openings were found to decrease both strength and rigidity of the wall as compared to a solid panel. The second series of tests was then designed to see if these influences could be modified by changes in the special steel around the openings.

The following conclusions were reported: (1) The behavior of walls containing openings can be predicted. (2) Application of the methods of strength of materials will produce results of sufficient accuracy for elastic design. (3) Long, diagonal well-anchored corner bars significantly increase both strength and rigidity of a wall containing openings once cracking begins.

Studies on Plastic Flow Characteristics of Lightweight Aggregate Concrete—T. J. McClellan, associate professor of civil engineering, Oregon State College, Corvallis

Information is needed on factors affecting plastic flow in lightweight expanded shale aggregate concrete, particularly since this aggregate is being used in precast prestressed concrete structural members. The material

was presented as a progress report since the program is only partially complete; only two mixes of varying ultimate stress level have been tested and no control mixes of conventional stone concrete have been loaded as yet.

It was determined that a procedure utilizing compression specimens rather than flexural specimens would be more likely to yield results directly applicable to the problem of stress loss in tendons of pretensioned members, particularly in the range of approximately 200 hr after stressing in which it appeared that the highest flow losses would be achieved.

No conclusions as to allowable magnitudes of plastic flow can be drawn at this time.

It is anticipated that ensuing tests will compare flow losses for conventional stone concrete with expanded shale aggregate concretes. It is proposed to continue the tests with varying mixes and at varying stress levels for each mix in an attempt to determine a pattern of stress loss for each mix.

Warping of Reinforced Concrete Due to Shrinkage—Alfred L. Miller, professor of mechanics and structures, University of Washington, Seattle

Objectionable deflections and deformations of structural elements, especially in relatively thin slabs and shallow beams of buildings, are often ascribed to faulty workmanship or materials, creep, and plastic flow when in fact they are evidence of warping. Warping due to temperature and moisture differentials is generally recognized, but the effect of shrinkage is uncertain.

Experimental investigation revealed that warping of reinforced concrete members due to shrinkage during the period of seasoning is an inherent characteristic of reinforced concrete that can be anticipated and controlled. A realistic theory was developed by which the amount of warping can be predicted and provision made for its reduction or elimination.

Steel reinforcement restricts but does not prevent shrinkage in its effective direction. It prevents free shrinkage of the concrete immediately adjacent to the steel to which it bonds, thereby inducing tension in the concrete and compression in the steel. The amount of concrete that participates in this interaction determines the compressive effect on the steel. When this interaction is eccentric with respect to the cross section, unsymmetrical strains cause warping.

The relationship between the shrinkage strain induced in the steel and the free shrinkage strain of the concrete after bonding is the principal factor that determines the warping of member of specified dimensions. This relationship was utilized along with the geometry of the curvature-strain relationship to predict warping deflections.

Warping can be eliminated by using symmetrical sections. It can be substantially reduced by a relatively small amount of reinforcement near the concave face. Low shrinkage concrete and a larger number of small bars rather than a smaller number of large bars can provide considerable reduction also.

SECOND TECHNICAL SESSION

Moderator for the second session was William A. Bugge, director, Washington State Highway Department, Olympia. The papers covered a variety of topics including colored and textured precast concrete units, reinforced masonry, high-density concrete, and a tunnel project in Vancouver.

Manufacture and Application of Colored and Textured Precast Concrete Units—Otto Buehner, Otto Buehner and Co., Ltd., Salt Lake City

Module sized units, intricate grill work, inlays, and other special applications are available in precast panels produced by the Mo-Sai method. A wide range of colors and textures have been made available. Technical aspects of casting, curing, and exposing colored aggregates were discussed in detail.

Reinforced Masonry As Developed in the Northwest—Verne Frese, president, Layrite Concrete Products of Seattle, Inc., Seattle

A review of developments in concrete masonry, particularly with application to the Pacific Northwest. Historically speaking, there were at least three developments of major importance to the concrete masonry field.

Peter Hostmark, center, was head of the group that planned the technical program at Seattle. A. W. LaGrelius, left, was in charge of banquet arrangements, and Ray Young, seated at the right, supervised the financial planning for the local committee

The first was the development of bond beam units and joint reinforcing. This horizontal reinforcing stiffened and restrained the walls and greatly reduced the cracks that occurred.

The second major factor was the recognition that drying of concrete was responsible for most of the shrinkage cracking and could be eliminated by reducing the moisture content of block before they were put in the wall.

The third significant step was the development of column block designs and other units to permit the efficient use of vertical steel.

In describing job applications in the Seattle area, it was noted that control joints are rarely used. The over-all structural integrity of a building is decreased by control joints, stated the author, and in an area where seismic forces must be considered this becomes an important factor.

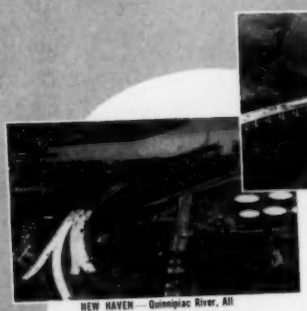
In addition to describing such details as bond beams, spandrel beams, and columns (some utilizing lightweight concrete units), joint reinforcing, the effect of various grades of mortar, waterproofing, painting, and cost were discussed.

High-Density Concrete for Shielding Atomic Energy Plants — Harold S. Davis, Hanford Atomic Products Operation, General Electric Co., Richland, Wash.

High-density concretes which have special compositions for improving shielding effectiveness may cost from $200 to $1000 per cu yd, including forms. Although these special concretes cost more than conventional concrete, their proper use may result in reducing the over-all cost of the project or in improving the efficiency of the operation.

Special concretes for constructing shielding structures are distinguished from conventional concrete by having (1) higher density and/or (2) special compositions to improve the attenuation of neutrons. In the design of shields for absorbing gamma rays, high density at minimum cost is of prime importance. In the design of concrete shields to attenuate neutrons, sufficient material of light atomic weight such as hydrogen should be included in the composition.

The optimum density and composition for a concrete shield are influenced by: (1) type and intensity of radiation, (2) limitations on



NEW HAVEN — Quinnipiac River, All Structural Concrete, D. B. Steinman of New York City

STRATFORD — Housatonic River, Tremie Concrete, Parsons, Brinckerhoff, Hall and Macdonald

BRIDGEPORT — Yellow Mill Pond, All Structural Concrete, E. Lionel Pavio of New York City

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NORWICH — Shetucket River, Tremie Concrete, John J. Mazzochi & Associates, Glastonbury, Conn. Ammann & Whitney, New York City

MILFORD — Overpass, Concrete Decks. Charles H. Sells of New York City

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AD 26-1

R. H. Lochow, head of the local Seattle committee, is flanked on the left by Lee Spahr who was in charge of ladies entertainment. Herbert Reed, right, assisted with transportation arrangements

shield dimensions or space restriction, (3) complexity of placing conditions and shape of forms, (4) number and proximity of through-tubes and embedded items, (5) proposed construction methods, (6) temperatures and operating conditions, (7) external and dead loads, (8) available aggregates, and (9) economy.

In general, mix proportioning for high-density concrete does not differ radically from that for conventional concrete; that is, the mix which produces the given quality of the hardened concrete at the least cost is usually the most desirable mix.

When working with a high-density concrete, the concrete laboratory is an essential part of the design and construction operation. It can be used effectively for confirming the adequacy of raw materials and procedures, and for indicating methods of cost reduction.

The Deas Island Tunnel—Norman D. Lee, Foundation of Canada Engineering Corp., Ltd., Vancouver, B. C.

One of the new Fraser River crossings, which will be a part of the Vancouver metropolitan freeway system, is the Deas Island Tunnel now under construction. There were several factors which combined to make a concrete tunnel more attractive than a steel bridge at the location chosen. Because of the deep, loose soil, the site was more suited to a tunnel; also, the tunnel places no piers in the river to hamper navigation. It was also possible to show a saving in first cost for a tunnel in preference to a bridge, due partially to the severe clearance requirements placed by navigation authorities and partially to the scheme of construction that was developed.

While the over-all length of the structure is about 4500 ft, the portion being constructed as a completely enclosed tunnel is 2165 ft. The tunnel portion will be made up of six precast elements, each weighing 18,000 tons. These tunnel elements were constructed in a drydock close to the site.

The tunnel is being excavated by the trench method. A special floating rig will be used to sink the precast tunnel elements into the trench. Hydraulic jacks built into the foundation blocks fixed to the underside of the elements will aid in positioning the tunnel units. Hydraulic jacks will also pull together the adjoining elements prior to making the permanent connection. Space under the tunnel will be filled by jetting sand into it; finally the trench will be backfilled and bed protection placed over the tunnel.

The rectangular tunnel cross section was designed to accommodate the normal loading of its own weight, water pressure, vertical rock and sand fill, lateral earth pressure, soil reaction and vertical frictional forces on the walls, and to resist earthquake loads. It has also been designed to withstand the flexural action which it will undergo as a beam on elastic supports since, due to the bed waves in the sand at the bottom of the river, there will be at times a variable load on the top of different sections of the tunnel. It has been estimated that these will result in maximum differences in the elevation of 3 to 4 in.

For the approaches, it was found that cantilevering the bottom slab to engage a greater mass of soil was the best way to obtain the required additional reaction against uplift.

Local committeemen—A. L. Miller, R. C. Hildebrandt, and C. E. Kitchin (reading from left) all of Seattle

THIRD TECHNICAL SESSION

Roy W. Morse, city engineer, Seattle, was moderator for the Wednesday morning session. Reports were devoted to the application of prestressing in shell structures, lift slab, and precast elements.

Prestressing of Cylindrical Concrete Shell Structures—John B. Skilling and John V. Christiansen, Worthington and Skilling, Seattle

Both prestressing and thin-shell construction have tended to free reinforced concrete from its traditional massiveness and have enabled architects and engineers to realize some of the economic and aesthetic possibilities of concrete structures, especially in long-span roof construction. The report was confined to the use of prestressing in cylindrical shells only. Post-tensioning strands may be placed in any of the basic elements of the cylindrical shell: shell, longitudinal edge beams, or transverse end stiffeners.

Strands may be placed longitudinally in the shell to good advantage. They may be draped in the shell from near the lower edge at center of span up into the diagonal tensile zone near the ends of the span, thus closely following the lines of principal tensile stress. However, from a practical standpoint this arrangement may be somewhat limited because of the thinness and curvature of the shell, and end anchorage in a thin slab may prove difficult.

Prestressing strands may be placed in the edge beams parallel to the axis of the shell. Although the curvature of the strands is limited by the depth of the beam, this arrangement has the practical advantage of providing a wider section in which to place the strands, limiting the curvature of the strands to one plane, and larger and fewer strands with conventional end anchorage can be used.

Prestressing strands may be placed in the transverse stiffeners at right angles to the axis of the shell. This may be of special value when the arrangement of stiffeners also results in large direct or flexural tension stresses in the stiffener.

Two structures recently completed in Washington were described to illustrate the use of prestressing with cylindrical concrete shell structures. A junior-senior high school in Yakima featured long-barrel cylindrical shells with prestressed edge beams over the auditorium and gymnasium. Eleven identical shell units, 16 ft wide and 2½ in. thick, were used to span 93 ft over the school auditorium. Five similar shell units, 41 ft wide and 3 in. thick, span 135 ft over the gymnasium.

The Boeing Airplane Co. hangar roof at Moses Lake was presented as an example of

cylindrical shells with prestressed transverse stiffeners. The building is 378 ft wide and 1076 ft long. Clear span of the airplane bays is 215 x 378 ft with a maximum height of 63 ft 1½ in. in the center. Fifty ft wide supporting shop bays are provided between the plane bays. The roof structure consisted entirely of 3 in. thick cylindrical shells stiffened by transverse arches and girders and longitudinal edge beams.

Economic Factors in Prestressed Lift-Slab Construction—Edward K. Rice, partner, T. Y. Lin & Associates, Los Angeles

To date, 41 structures involving 2,000,000 sq ft of prestressed lift-slabs have been constructed. The completed structures range in size and occupancy from 10,000 sq ft industrial buildings with roof parking to a six-story office building of 250,000 sq ft.

A major factor in the success of prestressed concrete lift-slab construction is that the prestressing and the lifting techniques complement each other. By prestressing the slabs, troublesome deflections experienced in reinforced slabs can be controlled, and by using the lift-slab construction method, the elastic and some of the plastic shortening of the slabs can conveniently be allowed for before the connection to the columns is made.

Not all structures can be framed economically with prestressed concrete lift slabs. To economically achieve satisfactory results with lift-slab spans greater than 18 ft, the slabs must generally be prestressed. Solid slabs are economical for spans of 20 to 30 ft. Bays 30 to 50 ft square are economical in prestressed cored lift slabs. Some types of structures which are not readily adaptable to prestressed lift-slab construction include structures on hillsides with stepped-grade slabs, structures which have a fairly close spacing of concrete walls which may be used as bearing walls, and structures of highly irregular shape. From the standpoint of total economy, the building layout and structural framing scheme is more critical in prestressed lift-slab construction than in other types of construction.

Various projects were illustrated to show construction techniques and the over-all utilization of prestressed lift slabs.

Design and Construction of a Precast and Prestressed Concrete Building for the Boeing Airplane Co. Developmental Center—Alfred T. Waidelich, vice-president, The Austin Co., Cleveland, and Arthur R. Anderson, Anderson, Birkeland and Anderson, Tacoma

The building described was a two-story structure, 540 x 560 ft. The second story was framed on a 40 x 60-ft module, and the first story on a 30 x 40-ft module. The framework consisted of precast concrete columns and prestressed concrete beams and girders. The roof deck was of precast lightweight concrete channel slab panels, and the exterior walls were prestressed lightweight concrete panels.

Continued on p. 21

E. J. Dale, left, took care of hotel arrangements for Seattle visitors; Fred Rhodes, center, handled registration and hospitality. Victor O. Gray, right, managed membership promotion efforts

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*Journal of the American Concrete Institute, June 1954.

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Continued from p. 19

The precast and prestressed concrete design was chosen because (1) the material was fireproof, (2) the construction schedule could be met, (3) the dead load was acceptable for the pile foundations, (4) a large paved area adjacent to the building was available for fabrication of the structural elements, and (5) repetition of typical structural members made it feasible to economically mass produce them of high-strength, prestressed and precast concrete.

The structure, in addition to the service load specified, had the additional requirement of resisting wind and seismic forces by flexure of the beams and columns. Thus the precast elements had to be joined in a manner to develop the equivalent of a monolithic structure having continuity and rigid joints. The building was designed to accommodate expansion and contraction by separating the structure with a double row of columns near the longitudinal and transverse building center lines, and connected together by expansion joints.

The exterior 5 in. thick walls were designed to carry their own weight from column to column. They spanned between 30 and 40 ft and varied up to 20 ft high. The panels were of expanded shale concrete with a 7-day compressive strength of 4000 psi. The 25- and 50-ft columns were cast from 5000-psi concrete. The 60-ft roof girders and 40-ft roof purlins were also 5000-psi concrete. The second-floor beams used 6000-psi concrete and were a combination pretensioned and post-tensioned design.

Erection of the thousands of precast and prestressed concrete members was completed within a few days of the scheduled time. This project successfully demonstrated the concept of joining pieces of prefabricated concrete and achieving the continuity of a monolithic structure.

INSPECTION TRIP, SPECIAL EVENTS

Wednesday afternoon was devoted to an inspection tour through the Boeing Developmental Center in Seattle. Although designed primarily as a research facility, the center serves a multiple function, as it could be used for production if the occasion arose. The Developmental Center occupies more than 40 acres of land and offers more than 1,000,000 sq ft of covered floor space. Many special provisions are incorporated for experimental and research programs, including vibration laboratories, sonic test cells, a structural test area where materials and structures will be tested to destruction, and underground laboratories with thick concrete walls and ceilings for tests of nuclear radiation. The plant will also provide engineering office facilities and ample shop space for fabrication of experimental parts and mock-ups.

Design and construction utilizing precast and prestressed concrete were described by Messrs. Waidelich and Anderson at the Wednesday morning session. The tour made it possible to observe the completed building with all elements described in the report viewed in relation to the over-all project which used more than 80,000 cu yd of concrete.

An amazing number of precast units were manufactured—1370 precast elements consisting of columns and post-tensioned beams and girders; 920 pretensioned beams; 209 precast, pretensioned wall panels, some as large as 20 x 40 ft; 6861–4 x 10-ft precast roof panels; and 4772–4 x 4-ft precast concrete panels for a work deck on the roof of the engineering building. Nineteen different mixes were used for the various applications of concrete on the project.

The 2-day meeting also featured one special luncheon and an evening banquet. The ladies' events included a tour of the city, fashion show, luncheons, and coffee hours.

LOCAL COMMITTEE MEMBERS

R. H. Lochow served as general chairman of the local committee in charge of the regional meeting; W. M. Lawton acted as secretary, and R. C. Hildebrandt was in charge of publicity and printing, assisted by Dan E. Seymour. All are with the Portland Cement Association in Seattle.

Fred H. Rhodes, Jr., University of Washington, Seattle, handled registration and hospitality, assisted by Trygve Bjornstad, Andersen-Bjornstad-Kane; Jack Clanton, University of Washington; Victor Gray, B. C. Olsen Co.; and Joseph G. Large, consulting structural engineer. Others assisting with registration included: Carter McClure,

Stevenson & Rubens; Holger Mittet, University of Washington; Lowell Northrop, Ideal Portland Cement Co.; Robert L. Pelton, Pacific Testing Laboratory; and Corwin A. Pilon, Washington Department of Highways.

Assisting Mr. Rhodes on the hospitality committee were: Harlan Edwards, Bindon & Wright; Harry Marshall, Marshall, Barr & Associates; Ed Strandberg, consulting structural engineer; and Fred Kettenring, Graystone Concrete Products, Inc.

The technical program was under the chairmanship of Peter Hostmark, Hostmark & Associates; members of his committee were Herman Adalist, North American Inspection & Testing; and Johann Enderleien, Charles P. Mayberg, and Bill Sellars, all of Peter Hostmark & Associates.

Hotel arrangements were taken care of by E. J. Dale, Portland Cement Association. Ray Young, Charles R. Watts Co., was in charge of the finance committee. The banquet committee was headed by A. W. LaGrelius, U. S. Navy Supply Depot, with Herman Adalist and Carter McClure assisting.

Herbert Reed, Shannon & Wilson, served as chairman of the transportation committee with Charles R. Bennett, Boeing Airplane Co., acting as vice-chairman; Donald E. Kemph, Washington Department of Highways, and Emile C. Mortier, Worthington & Skilling, were committee members.

Field trips were handled by Robert G. Haaker, Glacier Sand & Gravel Co. Membership promotion was in charge of Victor Gray, B. C. Olsen Co., and the committee for educational contact was headed by A. L. Miller, University of Washington.

Charles E. Kitchin, Decker, Christensen & Kitchin, served as chairman of the luncheon committee and Lee Spahr, Washington Concrete Products Association, headed the committee in charge of the ladies' entertainment.

Schaffner named associate dean

Charles E. Schaffner, formerly assistant dean, has been appointed associate dean of the Polytechnic Institute of Brooklyn. Prior to the appointment Dean Schaffner was professor of civil engineering and director of the evening session.

ACI Headquarters

Continued from p. 12

roof. Preliminary to plastering, glass fiber insulation with polyethylene vapor barrier was attached to the under surface of the folded plate. Metal lath was installed over this to support plaster. Fluorescent illumination will follow fold lines of the arresting roof surface; tubes will actually be mounted on projecting interior ridges of the plastered ceiling.

Building committee report

Most recent report from the ACI Building Committee shows the following contributions to the ACI Building Fund by categories, as of Nov. 30, 1957:

Cement producers	$ 74,212.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,176.22
Contractors	19,445.00
Ready-mixed concrete and aggregate industry	12,548.87
Membership at large	8,698.21
Admixtures manufacturers	7,458.93
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,570.00
Special contributions	1,766.43
Eastern Canada	1,496.90
	$205,957.56
Pledges in hand	4,740.00
GRAND TOTAL	$210,697.56

Goble granted Fulbright Scholarship

After receiving his MS from the University of Washington, George G. Goble was granted a Fulbright Scholarship to continue advanced concrete studies at the Stuttgart Technische Hochschule in Germany.

Goble's main interests are in prestressed concrete and thin shells. His present curriculum includes study of these subjects along with mathematics, advanced mechanics, structural design, and concrete properties. When he returns to this country he will continue his studies at the University of Washington, Seattle.

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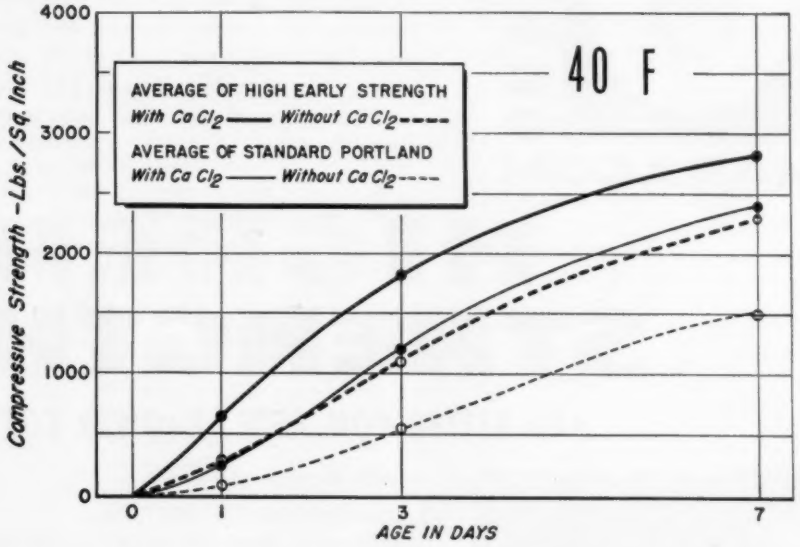

The American Concrete Institute now recommends the addition of calcium chloride for cold weather concreting. (ACI Standard "Recommended Practice for Winter Concreting.") These benefits have been proved, on jobsite after jobsite: initial and final set achieved up to three times faster, quicker early strength, greater ultimate strength. Finishing goes faster, forms can be pulled days earlier, contractor profit increases.

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Columbia Calcium Chloride is available in both High Test Flake (94-97% $CaCl_2$ content) and Regular Flake (77-80% $CaCl_2$). One 80 lb. bag of High Test delivers the same results as one 100 lb. bag of Regular. Both add easily at suppliers' plants or right at the jobsite.

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Positions and Projects

ACI Committee 214 name changed

The Institute's Board of Direction has approved a change in the title of Committee 214 to "Evaluation of Results of Strength Tests of Concrete." William A. Cordon, Utah State Agricultural College, Logan, heads the committee.

The word "compression" has been deleted from the committee title since future work planned for this group will include flexural as well as compression tests.

Philippine study group sees new ACI office building

A subcommittee on building construction of the Philippine Industrial Standardization Study, now touring the United States, visited the headquarters of the American Concrete Institute in November.

The group included: Cesar H. Concio, chairman of the study group, consulting architect and civil engineer; Gil O. Opiana, assistant director and research coordinator, Institute of Science and Technology, Manila; and Efren V. Mendoza, director of the Chamber of Commerce of the Philippines.

The standards of the American Concrete Institute were collected for consideration by the entire study group. The visitors were impressed by the international appeal of the architectural style of the new ACI headquarters in which they noted a "Sumatran influence."

Bergendoff receives award

Ruben N. Bergendoff, consulting engineer of Kansas City, Mo., was one of four engineers who received the Missouri Honor Award for distinguished service in engineering from the University of Missouri. A member of the firm of Howard, Needles, Tammen & Bergendoff, he was cited for "outstanding achievements in the design of long- and short-span bridges, valuable contributions to development of modern highway turnpikes, expressways, and interchanges, and untiring service in advancement of the engineering profession and to civic enterprises."

Upson marks 50 years with Raymond Concrete Pile

Maxwell M. Upson, distinguished as a foundation and harbor engineer, recently completed a half century of continuous service with Raymond Concrete Pile Co., New York. Mr. Upson joined the company in 1907 as secretary, general manager, and chief engineer. In 1930 he was named president and since 1946 has been chairman of the board.

He joined ACI in 1912 and was president of the Institute in 1926. He has been a member of ACI's Board of Direction, active in committee work, and a contributor of technical papers for JOURNAL publication. His patents, of which he holds more than 40, control many of the well-recognized methods used in placing of concrete piling and caisson foundations.

ERRATA—Title No. 51-32

"Design of Blast Resistant Construction for Atomic Explosions"

By C. S. Whitney, B. G. Anderson, and E. Cohen

ACI JOURNAL, March 1955, *Proceedings* V. 51

AVAILABLE WITHOUT CHARGE from ACI Publications Department

Hougland appointed field representative

Clifford P. Hougland has been appointed field representative for the American Concrete Pipe Association with headquarters in the Washington, D. C., office.

After receiving his engineering degree from Iowa State College, Mr. Hougland spent several years with the Iowa Highway Commission and Bethlehem Steel Corp. as engineer on several highway and steel construction projects. Since 1954 he has been active in sales promotion and public relations activities.

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WAX-RESIN BASE. A general purpose compound, usually used on mass concrete, pavements. Has excellent water retention properties, allowing slow curing of the concrete which eliminates checking. Dries rapidly and forms a film of uniform texture.

RESIN BASE. Recommended for buildings, offices, architectural concrete, etc. The pure resins, oils and other ingredients present a clear film which does not materially alter the appearance of concrete and it permits early application of adhesives, paint, etc.

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Pre-assembly of concrete pipe saves time on

Underwater Sewer Installation

Partial pre-assembly of reinforced concrete pipe sections resulted in substantial time savings during the recent construction of an 1100-ft interceptor sewer across the Allegheny River at Sharpsburg, Pa. The 30 in. diameter sewer, built for the Allegheny County Sanitary Authority sewage disposal system, was laid in a trench as much as 28 ft below the surface of the river; it will carry both sewage and surface water from the Greater Pittsburgh communities of Aspinwall, Sharpsburg, Blaw-Knox, Verona, and Montrose.

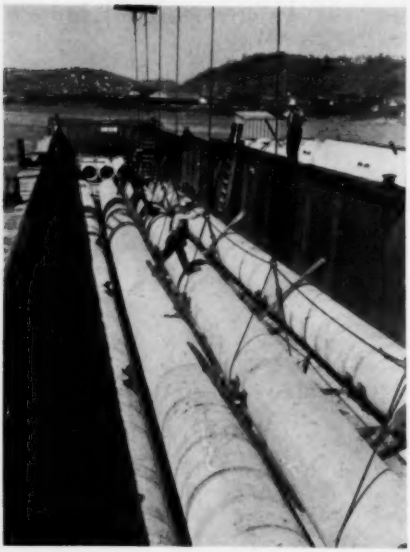
Since all connections in the interceptor sewer had to be made underwater by a diver, it was decided that the diver's underwater time could be substantially cut by reducing the number of connections. This was accomplished by pre-assembling ten 96-ft pipe sections, each consisting of six 16-ft sections.

About seventy 16-ft sections of the heavy-duty pipe were used in the entire project; the pipe has 5 in. thick walls, reinforced with steel mesh, and weighs 620 lb per ft. Pre-assembly of some of the king-size sections was made in the hopper barges which brought pipe to the site, but others were assembled after the 16-ft sections had been transferred to deck barges. Each connection was made by bolts passed through lugs set in the ends of the sections; joints were sealed with a rubber gasket.

A derrickboat, utilizing a special 4-ton rigging which supported the 30-ton pipe section in seven wire rope slings, swung the assembled section off the barge and lowered it into the trench in the river bed.

The diver then was lowered to the bottom of the trench, where he began issuing instructions to the derrickboat crew by telephone. With the assistance of a second derrickboat, the pipe was slowly shifted until its tapered end slipped into the recessed end

Derrickboat lowers a 96-ft length of concrete pipe into Allegheny River. Six 16-ft sections were pre-assembled to save underwater labor



A 30-ton section of concrete pipe 96 ft long is lifted from the hopper barge where it was assembled from 16-ft sections. Pipe was laid to form 30 in. diameter sewer under the Allegheny River

of the previously laid section and formed a watertight connection against the rubber gasket.

After bolting the new joint, the diver walked the length of the pile, backing off the bolts of the pre-assembled sections to allow for settlement. A pipe was then driven at the riverward end of the pipe and a cable from the pile looped around the pipe to hold it in position while a dredge backfilled the trench.

To allow sufficient fall to the trunk sewer, the interceptor was entrenched on a slope ranging from 16 to 28 ft below normal pool. This meant that, because of variations in the river bottom, the trench had to be excavated through from 7 to 26 ft of mud, gravel, and clay.

Access to the pipeline on the opposite bank is through a manhole structure being built by Harrison Construction Co., which subcontracted the pipeline work to Dravo Corp., Pittsburgh. Pipe sections were cast by Allegheny Concrete Pipe Associates.

Great Lakes Carbon opens mine

With the opening of its No Agua deposit of crude perlite in northern New Mexico as the first move, an extensive new expansion program is announced by the Perlite Department of Great Lakes Carbon Corp.'s Mining and Mineral Products Division, Los Angeles. To exploit this deposit, a new crushing and sizing plant is being erected at the site, scheduled for completion in June, 1958. The crushed and graded perlite ore will be sold to independent processors and users of perlite, as well as to franchisees of Great Lakes Carbon Corp.

German research men visit ACI

Representatives of the Huttenwerk Rheinhausen AG (Germany) visited headquarters of the American Concrete Institute recently to discuss requirements for reinforcing steel and prestressing steel in United States practice.

Walter Jaeniche, director of research and development, and Erdmann Stolte included this visit in a study of metallurgical problems concerning the steel industry in this country.

Kudroff named director of engineering

The appointment of Marvin J. Kudroff to the post of director of engineering of Daniel, Mann, Johnson, and Mendenhall, Architects and Engineers, Los Angeles, has been announced. Mr. Kudroff is an associate of the firm. A graduate of Pennsylvania State University, he has been with DMJM for more than 10 years as chief structural engineer and has been project manager of many of the organization's major jobs.

Chain Belt executive retires

J. C. Merwin has resigned from the board of directors of Chain Belt Co., Milwaukee. Mr. Merwin had been a board member for 34 years. During this time he also served as works manager, vice-president and treasurer, president, and chairman of the board.

L. H. Bosnian has been elected director to fill the vacancy created by Mr. Merwin's retirement. Mr. Bosnian, who has been with the company since 1919, was elected a senior vice-president in 1957.

Who's Who This Month

ACI Committee 323

"Tentative Recommendations for Prestressed Concrete," report of Joint ACI-ASCE Committee 323, Prestressed Reinforced Concrete, appears on p. 545. Thor Germundsson, Portland Cement Association, Chicago, is chairman of the committee; E. L. Erickson, Bureau of Public Roads, Washington, D. C., is vice-chairman; and W. B. Bennett, Jr., Portland Cement Association, Chicago, secretary.

Members of the committee include: Paul W. Abeles, consulting engineer, London; A. Amirikian, Bureau of Yards and Docks, Washington, D. C.; Raymond Archibald, J. E. Greiner Co., Seattle; Walter E. Blessey, Tulane University, New Orleans; William Dean, Florida Highway Department, Tallahassee; Curzon Dobell, Preload Enterprises, Inc., New York; Harry H. Edwards, Lakeland Engineering Associates, Inc., Lakeland, Fla.; and W. O. Everling, American Steel & Wire Co., Cleveland.

Other members of this 33-man committee are: Eugene Freyssinet, Societe Technique pour l'Utilisation de la Precontrainte, Paris; A. W. Hill, Cement and Concrete Association, London; Lloyd E. Hill, John A. Roebling's Sons Corp., Trenton, N. J.; Myle J. Holley, Jr., Massachusetts Institute of Technology, Cambridge; M. W. Huggins, University of Toronto, Toronto; Jack R. Janney, American Concrete Corp., Chicago; T. Y. Lin, University of California, Berkeley; N. M. Newmark, University of Illinois, Urbana; Gene M. Nordby, National Science Foundation, Washington, D. C.; Douglas E. Parsons, National Bureau of Standards, Washington, D. C.; G. S. Paxson, Oregon State Highway Department, Salem; and Howard F. Peckworth, American Concrete Pipe Association, Chicago.

Also serving on the committee are: Ervin Poulsen, civil engineer, Copenhagen; E. H. Praeger, Praeger-Kavanagh Engineers, New York; E. J. Ruble, Association of American Railroads, Chicago; Morris Schupack, The Preload Co., Inc., New York; C. P. Siess, University of Illinois, Urbana; Howard Simpson, Massachusetts Institute of Technology, Cambridge; Peter J. Verna, Jr., Concrete Materials, Inc., Charlotte, N. C.; W. R. Wilson, Atchison, Topeka & Santa Fe Railway, Chicago; R. F. Wittenmyer, Austin Co., Cleveland; and Charles C. Zollman, Charles C. Zollman and Associates, Newton Square, Pa.

At the time of his death in 1955, Albert E. Cummings was chairman of the committee, which had been under his leadership since 1949.

W. Sefton

W. Sefton, whose paper "Multistory Lift-Slab Construction" appears on p. 579, is president of W. Sefton & Associates Ltd., consulting engineers of Toronto. He graduated from the University of London in 1938 and since then, except for 6 year's service in the Royal Engineers during the war, he has been responsible for reinforced concrete and structural steel design of all kinds.

Recent projects include the 22-story Hotel Carlton in Toronto, the Vancouver Civic Auditorium in British Columbia, and the Canadian Pavilion for the International Fair in Brussels, Belgium. He has also designed a number of lift-slab projects in Canada, where he introduced structural steel fabricated collars in place of the cast steel collars formerly used exclusively.

R. H. Heiskell, R. H. Black, R. J. Crew, and H. Lee

The results of a series of experiments on "Elevated Temperatures of Portland Cement Mixtures Related to Surface Removal" compiled by R. H. Heiskell, R. H. Black, R. J Crew, and H. Lee appears on p. 591.

R. H. Heiskell is a member of the Technical Developments Branch, U. S. Naval Radiological Defense Laboratory, San Francisco. He has held his present position since 1950

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and has worked for several years with the problem of decontaminating concrete surfaces—a project from which this paper evolved. In 1953 he was sent to Chalk River, Canada, to work out procedures for the decontamination of concrete surfaces contaminated during a nuclear reactor accident. During recent years Mr. Heiskell has conducted studies to minimize the problem of decontaminating concrete through the use of suitable protective coatings.

Graduating from Oklahoma State University in 1942, Mr. Heiskell was employed by the Naval Ordnance Laboratory in Washington, D. C., from 1942 to 1946. For the next few years before assuming his present duties, he was half-owner of an industrial testing and research laboratory, concentrating principally on concrete and concrete forms.

R. H. Black, on graduating from the University of California in 1950, joined the U. S. Naval Radiological Defense Laboratory in San Francisco. During his research there he has determined the degree of radiological fallout contamination of various materials and surfaces and evaluated methods for reducing contamination and/or facilitating decontamination. A part of this work involved developing means for the decontamination of concrete. These decontamination methods were utilized in the recovery of the NRX reactor, Chalk River, Ontario, in 1953.

He has assisted in the evaluation of protective coatings for concrete, and in feasibility studies for the large scale decontamination of concrete pavements.

R. J. Crew graduated from Cornell University in 1939. He then joined the Shell Development Co. as a chemical engineer. There he worked on the pilot plant scale development of alkylation, isomerization, and hydrogenation processes for the production of high octane fuels and additives. Mr. Crew developed an esterification process and supervised the pilot plant operation for the production of synthetic resins. He also supervised the production of corrosion inhibitors and various organic chemical products while with the Shell organization.

He began work with the U. S. Naval Radiological Defense Laboratory in 1950 in the chemical technology division. He has developed procedures for the radiological decontamination of various surfaces and building materials. He also assisted in the decontamination of the nuclear reactor building of the Atomic Energy of Canada, Ltd., at Chalk River. Mr. Crew has developed procedures to determine the effectiveness of protective coatings on concrete and their ability to facilitate the decontamination of concrete and mortar surfaces.

Hong Lee, the fourth contributing author of the group, graduated from the University of California in 1950, with a BS in mechanical engineering. He joined the U. S. Naval Radiological Defense Laboratory in 1950, and there at the present Mr. Lee is engaged in radiological countermeasures research.

George H. Nelson and Otto C. Frei

"Lightweight Structural Concrete Proportioning and Control," p. 605, has been prepared by George H. Nelson and Otto C. Frei.

George H. Nelson graduated from the University of Illinois in 1932 and for the past 24 years has been in the independent laboratory field. At the present he is president of Law Engineering Testing Co. of Atlanta. He has specialized in concrete technology in the laboratory and in production, and has supervised testing and inspection of materials on many of the largest industrial installations in the country.

Active in ACI committee work, Mr. Nelson is chairman of Committee 401, Guide to Specifications for Structural Concrete; he is a member of Committee 605, Hot Weather Concreting, and Committee 613, Recommended Practice for Proportioning Concrete Mixes. He is the author of numerous technical papers, but this is his first contribution to the ACI JOURNAL.

Otto C. Frei is president of Georgia Lightweight Aggregate Co. in Atlanta. He began his career in the manufacture of portland cement as analytical chemist for Pacific Portland Cement Co. and the Yosemite Portland Cement Corp. In 1936 he became chief chemist for Washington-Idaho Lime Products Co., Spokane, and in 1939 was elected president of the corporation. Later

he served as president of the Columbia Portland Cement Co., also in Spokane.

Mr. Frei became active in the design and operation of lightweight aggregate plants using the rotary kiln production method in 1948. Since then he has acted as consultant in this field on plant design, equipment selection, operation procedures, and lightweight concrete technology. He is a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete.

Huron announces promotions

Blaine E. Miller has been named division sales manager, Ohio and Michigan Division, Huron Portland Cement Co., Detroit.

Other promotions include that of Robert E. Brown to district sales manager of the Toledo district; Ralph F. Dickson to division sales manager, New York and Wisconsin Division, and Jack Giles to assistant traffic manager. Dickson and Giles will both work in the company's Detroit office.

LOOKING AHEAD

January 26-30, 1958—Associated Equipment Distributors, 39th Annual Meeting, Conrad Hilton Hotel, Chicago.

January 27-30, 1958—Plant Maintenance and Engineering Show and Conference, Amphitheatre and Palmer House, Chicago, Ill.

January 27-31, 1958—University of Florida–Prestressed Concrete Institute, Second National Prestressed Concrete Short Course, Ellinor Village, Daytona Beach, Fla.

February 9-12, 1958—Mason Contractors Association of America, 8th Annual Convention and Show, Sheraton-Park Hotel, Washington, D. C.

February 10-13, 1958—National Ready Mixed Concrete Association, 28th Annual Convention, Conrad Hilton Hotel, Chicago, Ill.

February 10-13, 1958—National Sand and Gravel Association, 42nd annual convention, Conrad Hilton Hotel, Chicago, Ill.

February 10-14, 1958—American Society for Testing Materials, Committee Week, Hotel Statler, St. Louis, Mo.

February 17-19, 1958—National Concrete Masonry Association, 38th Annual Convention, Chicago, Ill.

February 17-19, 1958—National Crushed Stone Association, 41st Annual Convention and Biennial Exposition of the Manufacturers Division, Hilton Hotel, Chicago, Ill.

February 24-26, 1958—National Concrete Products Association (Canada), Ninth Annual Convention, Seigniory Club, Montebello, Quebec.

February 24-26, 1958—National Concrete Contractors Association, 1st Annual Convention and 1958 Concrete Exposition, Chicago Coliseum, Chicago, Ill.

February 24-27, 1958—ACI 54th Annual Convention, Morrison Hotel, Chicago, Ill.

February 24-28, 1958—American Society of Civil Engineers, National Convention, Sherman Hotel, Chicago, Ill.

Honor Roll

February 1—December 31, 1957

Blas Lamberti proposed 10 new members this month putting him in first place on the Honor Roll for 1957 with 25 credits. G. B. Southworth is in second place with 22 credits, Ernst Maag is in third place with 14 and Carl H. Koontz is fourth with 12 credits.

Beginning with this issue of the JOURNAL the Honor Roll year will end December 31 each year and will include all credits from January 1 through December 31 of that year. The 1957 Honor Roll below represents only 11 months of membership activity—thus making effective the change which coordinates Honor Roll year with calendar year.

The "member getting" efforts of the following ACI members have given us a net gain of over 800 new members in the last year. We hope that you will keep up the good work and make our next Honor Roll year better than ever.

Blas Lamberti....25
G. B. Southworth....22
Ernst Maag....14
Carl H. Koontz....12
Nicandro Barboza....10
Phil M. Ferguson....10
Arthur N. L. Chiu.... 9
H. C. Pfannkuche.... 9
Martin J. Gutzwiller.... 8
Yesh Etkin.... 7
I. Martin.... 7
Luis Perez Cid.... 6
Jaime de las Casas.... 6
Ambrosio R. Flores.... 6
Walter H. Price.... 6
Samuel Hobbs.... 5
Myle J. Holley, Jr.... 5
L. M. Legatski.... 5
William McGuire.... 5
Sabri Sami.... 5
James E. Amrhein.... 4
W. H. Armstrong.... 4
W. S. Cottingham.... 4
George A. Dinsmore.... 4
Louis A. Gottheil.... 4
J. E. Jellick.... 4
Franklin B. Johnson.... 4
Henry L. Kennedy.... 4
Daniel S. Ling.... 4
Robert H. Lochow.... 4
James A. McCarthy.... 4
Frederick T. Mavis.... 4
Jerome M. Raphael.... 4
Martin Schulz.... 4
Roberto Zepeda.... 4
S. E. Acker.... 3
Luther E. Bell.... 3
George F. Bishop.... 3
R. F. Blanks.... 3
F. Thomas Collins.... 3
William V. Coyle.... 3
Roger D. de Cossio.... 3
C. A. Engman, Jr.... 3
Andrew G. Fallat.... 3
Thomas C. Kavanagh.... 3
G. M. Kidd.... 3
R. A. Kirkpatrick.... 3
Leo Liberthson.... 3
Sam C. McCluer.... 3
Gerald Milsom.... 3
George H. Nelson.... 3
Abdur Rahman S. Rasul.... 3
Theodore O. Reyhner.... 3
Wallace W. Sanders, Jr.... 3
William C. Schwenger.... 3
Seymour K. Shapiro.... 3
Howard Simpson.... 3
Ellis S. Vieser.... 3
J. F. Weigel.... 3
H. T. Williams.... 3
John T. Young.... 3

Miles N. Clair....2½
Alfred Y. Aragaki....2
Adolfo Arango....2
Mark J. Baron....2
G. M. Brief....2
George Brockway....2
Carlos D. Bullock....2
Jose L. Capacete....2
James Chinn....2
Dale Cobb....2
George D. Crocker....2
C. F. DeVilbiss....2
B. M. Dornblatt....2
Harlan H. Edwards....2
P. E. Ellen....2
E. I. Fiesenheiser....2
Kenneth H. Gedney....2
Herbert J. Gilkey....2
John L. Goetz....2
Victor O. Gray....2
George C. Guibert....2
Albert Haertlein....2
Roger E. Harlepp....2
Will M. Heiser....2
N. L. Hinkson....2
Peter S. Hobdan....2
John J. Hogan....2
M. W. Huggins....2
Truman R. Jones, Jr....2
H. J. Keenan....2
Clyde E. Kesler....2
F. R. Khan....2
Yavuz Kirec....2
Arthur F. Kirstein....2
Thorton Kline....2
L. Knight....2
Oscar Latorre M....2
Harry McLean....2
Patrick McNally....2
A. J. Macchi....2
Ronald A. Maina....2
Neal B. Mitchell, Jr....2
Izuo Miyashita....2
Otto H. Monch....2
John M. Muir....2
D. F. Nash....2
George Papoulas....2
Ramiro Parada....2
Richard A. Parmelee....2
Frederick A. Paterson....2
Jorge Prince....2
Harold Schweitzer....2
Marcel V. Sous de C....2
Mete A. Sozen....2
Joaquin Spinel L....2
C. P. Thompson....2
E. W. Thorson....2
Joseph J. Waddell....2
James L. Watson....2
Robert A. Williams....2
R. A. Williamson....2
L. Willoughby....2
D. K. Woodin....2
Frank Kerekes....1½
Julius G. Ortiz, Jr....1½
C. H. Scholer....1½
P. W. Abeles....1
Robert F. Adams....1
Michael Alexander....1

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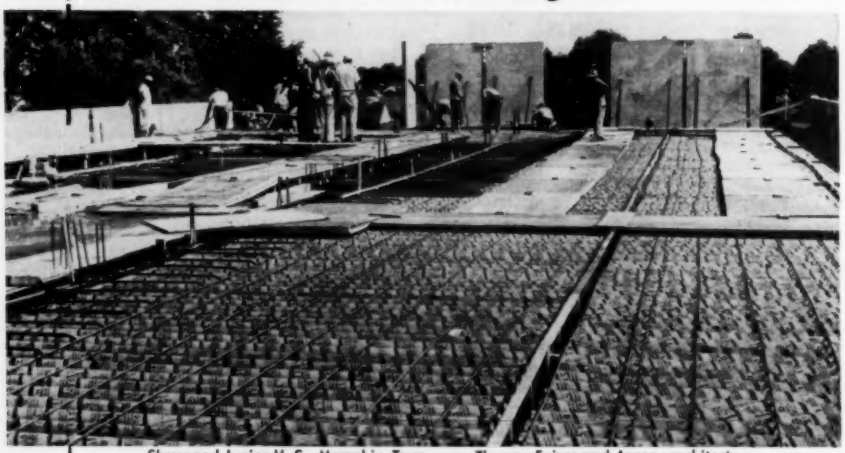

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1046

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J. M. Arboleda D....1
Ara Arman....1
Haraldur Asgeirsson....1
Donald S. Austin....1
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Angel Herrera Blanco....1
Walter E. Blessey....1
Major G. Bliss....1
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E. K. Borchard....1
W. E. Bradbury....1
Robert J. Brandt....1
John E. Breen....1
Francis Brodigan....1
Howard W. Brown....1
W. S. Burke....1
S. L. Burns....1
Charles M. Butler....1
Nicholas Bykowsky....1
Robert A. Cameron....1
G. A. Campbell....1
M. E. Capouch....1
Leo W. Charlstrom....1
Brian R. Case....1
A. B. B. Cassidy....1
T. J. Cavanagh....1
Washington T. Char....1
T. Z. Chastain....1
Mauricio Chedraui....1
Constantine Chekene....1
Howard F. Christensen....1
John V. Christianson....1
M. D. Cohan....1
Felix Colinas....1
Bernardo Contreras S....1
H. W. Cormack....1
Leo H. Corning....1
Manus Corvanich....1
William C. Craig....1
M. A. Craven....1
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Wayne E. Duncan....1
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Thor Germundsson....1
Edgar Gilbert....1
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Martin Goldstein....1
E. Gonzalez Rubio....1
H. F. Gonnerman....1
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Loyd Green....1
Samuel J. Green....1
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Alf Grini....1
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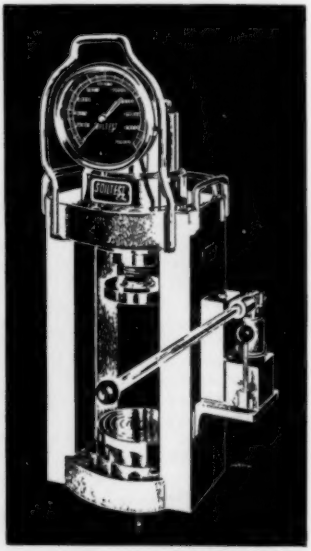

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Chris C. Ziogas.....1
A. Allan Bates..... ½
Robert A. Caughey..... ½
R. E. Copeland..... ½
Omer A. Fettahlioglu..... ½
Cornie Leonard Hulsbos..... ½
Edward Mangotich..... ½
James D. Piper..... ½
Charles Wuerpel..... ½

New Members

The Board of Direction approved 104 Individual applications, 2 Corporations, 20 Juniors, and 46 Students, making a total of 172 new members. Considering losses due to resignations, deaths, and nonpayment of dues, the total membership on Dec. 1, 1957, was 9172.

Individual

AMIR, DAVID, KIBBUZ ARZI, Tel Aviv, Israel (C. E. & Arch., Machleka Technitch)

ANDERSON, WILLIAM J., Paramus, N. J. (Vice-Pres. & Chf. Engr., Samworth-Hughes Co.)

ARAS, RESTAN M., New York, N. Y. (Struct. Engr., Bridge Design, D. B. Steinman)

ARAUJO C. OVIDIO E., Indianapolis, Ind. (Conc. Bridge Designer in Trng., State Hwy. Dept. of Indiana)

AUSTIN, WALTER J., Urbana, Ill. (Assoc. Prof., Univ. of Ill.)

BARRETT, CLYDE E., Concrete, Wash. (Sr. Struct. Inspector, Stone & Webster Corp.)

BESTON, JAMES A., Port of Spain, Trinidad, B. W. I. (Struct. Designer, Mence & Moore)

BLAIR, W. ROBERT, Los Angeles, Calif. (Mgr., Contr. Div., Sales Dept., Southwestern Portland Cement Co.)

BOLTON, JOHN, London, England (Cons. Engr., Civil & Struct. Engrg.)

BONUS, ROBERT M., Detroit, Mich. (Pres., Hay-Con Tile Co.)

BOTELLA, OVIDIO, Mexico, D. F., Mexico (Arch.)

BOWERS, WILLIAM S., Seattle, Wash. (In charge of Constr., Wash. & Alaska, Oil Equipment Service, Inc.)

BUCKLEY, F. L., Ft. Walton Beach, Fla. (Supv. Struct. Engr., U. S. Air Force)

CONCIO, CESAR H., Manila, Philippines (Arch., Engr., C. H. Concio & Assocs.)

COREY, BRUCE M., Burlington, Ont., Canada (Supv. of Civil Engrg., Steel Co. of Canada)

CRAWFORD, RICHARD L., Guatemala City, Guatemala (Res. Cons. Engr., Tippetts-Abbett-McCarthy-Stratton, Gibbs & Hill)

DAGSETH, ERIC LIND, Hollywood, Calif. (Chf. Engr., C.H.A.D.-J.V.)

DALE, VERN C., Holladay, Utah (Field Engr., Master Builders Co.)

DAVILA, SAMUEL, Lima, Peru (Tech. Advisor, Concretera Lima)

DAYTZ, ALEXANDER B., Los Angeles, Calif. (Sr. Struct. Engr., State of Calif., Div. of Arch. Schools)

DE ABREU, PERCY LOUZADA, Porto Alegre, RGS, Brasil (C. E.)

DE PICCIOTTO, MAURICE, New York, N. Y. (Struct. Designer, Marine Terminal Facilities, The Port of N. Y. Authority)

DIMITRIEF, ALEXIS, Montreal, P. Q., Canada (Struct. Designer, Henry Jasen, Ltd.)

EL-RAWI, SALMAN M., Philadelphia, Pa. (Struct. Engr., Richardson & Gordon Assocs.)

FAELTEN, EDGAR R., Buffalo, N. Y. (Cons. Engr., Civil & Struct.)

FENTON, J. GORDON, Spokane, Wash. (Engr., Central Pre-Mix Concrete)

FINLEY, WILLIAM W., JR., South River, N. J. (Constr. Engr., C. E., Burns & Roe, Inc.)

FISHER, WAYNE E., New Kensington, Pa. (Sales Mgr., Penn Builders Supply)

FUBINI, GINO, Great Neck, N. Y. (Builder & C. E.)

GALLETTA, NUNZIO, Pittsburgh, Pa. (Foundation & Struct. Design)

GERFEN, W. HOWARD, San Marino, Calif. (Cons. Struct. Engr.)

GOHMAN, LEWIS W., West Palm Beach, Fla. (Asst. Mgr., C. B. S. Div., Maule Ind.)

GRAHAM, V. B., Austin, Tex. (Mgr., Prestressed Div., Texas Quarries, Inc.)

GRAY, EDWARD D., Waukesha, Wis. (Exec. Vice-Pres., Great Lakes Lift Slab Corp.)

GULLAKSEN, JOHN R., Chicago, Ill. (Struct. Engr.)

HANSON, S. G., Detroit, Mich. (Proj. Mgr., Herbert Realty)

HARDING, JOHN F., Winslow, Wash. (Sales Engr., Master Builders Co.)

HARRIS, MILTON E., London, Ont., Canada (Gen. Mgr., J. Harris & Sons, Ltd.)

HARTZELL, FRANK C., JR., New York, N. Y. (Designer, Struct., Moran, Proctor, Mueser & Rutledge)

HAUSWALD, ARTHUR C., Elmhurst, Ill. (Struct. Engr., A. Epstein & Sons)

HO, TSO SHING, Kaohsiung, Taiwan, China (Deputy Dir. of Bureau of Engrg. Services, Ministry of Economic Affairs, Republic of China)

HOLSTEIN, JOHN J., Los Angeles, Calif. (Struct. Engr., Owner)

HOOKINGS, ROBERT S., Aurora, Ont., Canada (Proj. Engr., Pearce Constr. Co., Ltd.)

HOOVER, C. DONALD, Whittier, Calif. (Chf. Engr., Ropp & Ropp)

HOSOI, JUNZO, Tokyo, Japan, (Director, Nihon Cement Co.)

HOUSNER, GEORGE W., Calif. Inst. of Tech., Pasadena, Calif. (Prof. of Civil Engrg.)

HOWELL, STANDLEY, Norfolk, Va. (Mgr., Specifications & Estimates Branch, District Pub. Wks. Office)

HROMADIK, JOSEPH J., Oxnard, Calif. (Sr. Proj. Engr., U. S. Naval Civil Engrg. Lab.)

HUCK, ROBERT L., Baton Rouge, La. (Chf. Struct. Engr., Bodman & Murrell & Smith)

JOHNSON, GEORGE A., JR., Charleston, S. C. (Cons., Struct. Engr.)

JONES, ROBERT E., San Jose, Calif. (Struct. Engr.)

KARDESTUNCER, HAYRETTIN, Newark, N. J. (Struct. Designer, Porter, Urquhart, McCreary & O'Brien)

KHALIFA, H. S., Chicago, Ill. (Struct. & Civil Engr., City of Chicago)

KONDO, ESAKU, Naha City, Okinawa (Arch., Caltex (Overseas) Ltd.)

KU, CHIA CHENG, Bronx, N. Y. (Struct. Designer, Moran, Proctor, Mueser & Rutledge)

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LESSEL, ARTHUR G., Toronto, Ont., Canada (Spec. Writer, Wallace-Carruthers & Assocs. Ltd.)
LEVIEN, ROBERT E., Seattle, Wash. (Struct. Designer, Worthington & Skilling)
LINGO, DONALD E., Cleveland, Ohio (C. E., Ernest F. Donley's Sons, Inc.)
LOCKHART, D. H., St. Catharines, Ont., Canada (Office Mgr., Prod. Supt.)
LOFTUS, JOHN THOMAS, Brooklyn, N. Y. (Struct. Engr., Brown & Blauvelt, Cons. Engrs.)
LUBETKIN, BERNARD M., Maracaibo, Venezuela (Area Engr., Deep Water Concessions, Mene Grande Oil Co.)
MARCHESE, CHARLES, Oxnard, Calif. (Sr. Field Engr., Bechtel Corp.)
MCDONALD, JAMES B., Pittsburgh, Pa. (Assoc. Prof., Univ. of Pittsburgh & Private Cons. Practice)
MOE, JOHANNES, Skokie, Ill. (C. E., Lie Tech., PCA)
NORTON, LAWRENCE H., Rosemount, Montreal, Que., Canada (Concrete Co-ordinator, The St. Lawrence Seaway Authority)
OLTMANS, O. H., Monterey Park, Calif. (Sec.-Treas., Gen. Contr.)
PARSONS, JAMES STEWART, Cottom, Ont., Canada (Insp., Dept. of Hwys.)
PERLMAN, HYMAN, Brooklyn, N. Y. (Struct. Engr., Moran, Proctor, Mueser & Rutledge)
PETREY, HARRY G., Alhambra, Calif. (Struct. Design, Partner, Petrey & Knowles, Struct. & Civil Engrs.)
PRATT, ALBERT S., JR., LaJolla, Calif. (Struct. Engr.)
RIOS, ALBERTO, Bogota, Colombia (Design Engr., OLAP)
RULE, RHODES E., Los Angeles, Calif. (Arch. & Engr.)
RYDER, WILLIAM C., Chicago, Ill. (C. E., Struct. Design of Filtration Plants, Dept. of Pub. Wks., Bureau of Engrg., City of Chicago)
SAPPER, EUGENE H., San Diego, Calif. (Sales Repres., Southwestern Portland Cement Co.)
SCOTT, HAROLD A., North Vancouver, B. C., Canada (B. C. Respres., The Master Builders Co., Ltd.)
SCOTT, W. E., Tampa, Fla. (Mgr., Hartstone Concrete Products Co., Inc.)
SHEFFET, JOSEPH, Los Angeles, Calif. (Cons. Struct. Engr.)
SILLIMAN, CAPT. JULIAN W., Philadelphia, Pa. (CEC, USN)
SQUIRE, ALBERT T., Rochester, N. Y. (Engr. in charge of electrical & mechanical design, The Lozier Co.)
STRAUSS, MAX W., Beverly Hills, Calif. (Asst. Supt. of Bldgs., Bldgs. Dept., City of Beverly Hills)
THOMASS, L. N., Auckland, New Zealand (Sr. Lecturer in Civil Engrg., Auckland Univ. College)
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Junior

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HARRINGTON, DANIEL E., Flushing, N. Y. (Designer, Moron, Proctor, Mueser & Rutledge)
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TOLEDO, JORGE, Champaign, Ill. (Univ. of Ill.)
VENKAYYA, VIPPERLA B., Columbia, Mo. (Univ. of Mo.)
YTTERBERG, ROBERT F., Crestwood, N. Y. (MIT)

Tools, Materials, Services

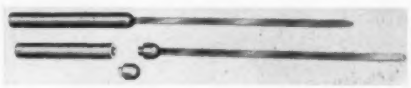
Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

1957 compilation of rental rates for construction equipment

Associated Equipment Distributors has announced that the 1957 (ninth edition) "Compilation of Rental Rates for Construction Equipment" is now available to the construction industry. It contains over 80 pages of national average rental rates for approximately 2000 items of construction machinery, and illustrates many of the major types of equipment. Rental rates were established by an AED survey conducted during May and June, 1957, among the association's 700 distributor members in the United States. $5 per copy. —Associated Equipment Distributors, 30 East Cedar St., Chicago, Ill.

Slender blades to vibrate concrete

Slender blades designed to vibrate concrete in extremely confined areas come in varying lengths, attach readily to vibrators. Model 1005S Sabre (22 in. long) is 1¾ in. wide x $\frac{1}{4}$ in. thick; Model L-10S (20 in. long) is 1¼ in. wide x ¼ in. thick. Light in weight, they are engineered of high grade spring steel



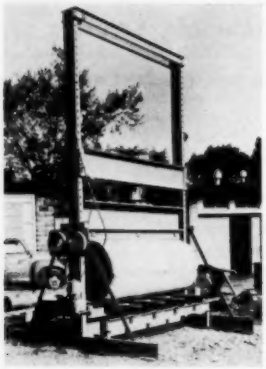
for long lift. Operating up to 10,000 rpm at high cycle or 14,000 rpm at 60 cycle, the "Sabre" blades easily offer full force, amplitude, and frequency for close work in prestressed concrete or in restricted areas where normal vibrators cannot operate, according to the manufacturer.—Dart Manufacturing Co., Denver, Colo.

Pin-up maintenance chart for truck mixers

Display chart applying to all makes of mixers, outlining 16 steps for complete maintenance from hold-down bolts to water pumps, is identified as Bulletin No. G-2486P. Chart can be posted on garage walls and other conspicuous places calling operator's attention to both periodic and daily maintenance practices to cut down repair costs.—Worthington Corp., Advertising & Sales Promotion Dept., Harrison, N. J.

Winter concreting objectives aided by calcium chloride

An 8-page brochure describes in brief form the "Recommended Practice for Winter Concreting (ACI 604-56)" adopted as a standard by the American Concrete Institute. Of particular interest are the eight charts which show the comparative compressive strength of concrete made with Type I and Type III cements with 0 and 2 percent calcium chloride by weight of cement. Temperature comparisons are made at 25, 40, 55, and 73 F.—Calcium Chloride Institute, 909 Ring Bldg., Washington 6, D. C.



Pipe tester

For testing concrete pipe up to 84 in. inside diameter in 8-ft lengths, the standard PT-75 pipe tester is equipped with both manual and electric pumps, and two 12 in. diameter pressure gages. This model has a capacity of 150,000 lb ram pressure and will also handle smaller pipe down to 6 in. inside diameter; the machine can be modified to handle pipe as large as 12 in. diameter in 16-ft lengths. The frame is of knockdown construction for easy erection at different job sites.

The pressure beams are engineered for minimum deflection at the maximum pressure of 150,000 lb and the upper beams are raised and lowered by means of a hand operated winch. Suspended weight is counterbalanced with a heavy coil spring.

"Package Unit" consisting of the gages and complete hydraulic apparatus is available for use on existing frames, or for inspection purposes. Described in Bulletin PT-75.—Forney's Inc., Tester Div., P. O. Box 310, New Castle, Pa.

Concrete forming guide

A 40-page booklet, "Your Pocket Guide to Successful Forming," explains economical ways to build and strip forms. The booklet gives practical information on the use of sheathing as well as plywood panels for forming, and covers rate-of-placement, tie spacing, and a technique for radius-wall form construction. Detail illustrations of form hardware and instructions for its use are designed to serve as a handy, on-the-job source or reference.—Gates & Sons, Inc., 80 S. Galapago, Denver 23, Colo.

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Weight measurement and control bulletin

Bulletin 12, "Weight . . . It's Measurement and Control," is the first comprehensive coverage of instrumentized weighing of bulk material, according to its publisher. The 20-page bulletin compares batch-in, batch-out, and continuous weighing processes, and explains how a completely automatic weighing system can be assembled by building-block techniques using unitized weighing components. Both pneumatic and electrical weight transmitters are discussed. Included is a description of the company's line of conveyor scales and weigher-feeders which can be added to existing systems.—Weighing and Control Components, Inc., 306 Lincoln Ave., Hatboro, Pa.

Water-thin coating for concrete forms

A coating for concrete forms (metal, plastic, or wood), which can be used with any concrete mix, according to the manufacturer, has been named Formgard; it will gradually replace Eziform now being marketed. In citing the advantages of this improved coating the manufacturer states that it dries immediately upon application; completely waterproofs the form face; can be hand or power-sprayed, or applied with a brush or mop; is economical. Comes in a concentrated form to be mixed with four parts of regular furnace fuel oil.—Industrial Synthetics Corp., 2000 W. Walnut St., Chicago, Ill.

Rotary kiln

Breaking sharply with traditional manufacture of rotary kilns, Vulcan Iron Works has developed a unit suitable for inclusion in any plan of automation where a rotary kiln is employed.

All kiln trunnion and thrust rollers have been linked to girth gears, cancelling out dependence on friction to move these rollers. Through such linkage, all rotating parts are synchronized to rotate simultaneously without slippage. This design consolidates running gear components, reducing the number of supporting piers and cutting cost. Heavy castings have been replaced with strong, lightweight fabricated parts machined to close tolerances. All are enclosed within sealed housing. Design permits forced feed lubrication of all gear teeth and bearing surfaces.—Vulcan Iron Works, Wilkes-Barre, Pa.

Concrete column form design calculator

A pocket calculator designed to answer troublesome problems of form design on columns and piers up to 48 in. sq or 65 in. diameter, including rectangular

cross sections, is now available. This easy-to-use calculator specifies the correct lumber sizes, the arrangement and spacing of the bracing members, and the proper size and spacing of the steel strapping which replaces clamps in these designs. Price $1.00.—Signode Steel Strapping Co., Calculator Dept., 2600 North Western Ave., Chicago, Ill.

Concrete carriers

Illustrated brochure describing four-wheel and six-wheel drive concrete carriers includes detailed specifications on brakes, engine, load capacity, and dimensions for five models. Choice of three four-wheel drive carriers: Model 50-50 designed to carry 36,000 lb gross vehicle weight; Model 45-55 and Model 45-55-JT, which have a 42,000-lb gross vehicle weight; two six-wheel drive units: Model 18-34, carrying 52,000-lb gross, and Model 18-28, carrying 56,000 lb gross vehicle weight.

On-the-job photos illustrate the all-climate applicability of these concrete carriers. Cantilevered engine is accessible for servicing; 144-in. and 155-in. wheel bases provide shorter turning circle, according to producer.—Oshkosh Motor Truck, Inc., Oshkosh, Wis.

Ball penetration apparatus

The instrument illustrated for determining the consistency of fresh concrete is made in accordance with ASTM C-360, "Ball Penetration in Fresh Portland Cement Concrete." It consists of a cylinder with ball-shaped bottom and a handle weighing 30 lbs. A lightweight metal frame guides the handle and serves as a reference for measuring the depth of penetration of the ball. Two times the ball reading is approximately equal to the slump of concrete.—Soiltest, Inc., 4711 W. North Ave., Chicago, Ill.

Conveyor catalog

A 36-page catalog describes job-engineered conveyors for special applications of length, capacity, horsepower, or other conditions where a pre-engineered conveyor is not suitable, and a line of pre-engineered (packaged) conveyors.

Much attention has been given to conveyor accessories such as the three drive types: torque-arm reducer; V-belt and countershaft; and gear-motor reducer. Specifications are furnished for extension frame sections (head and tail); channel frame conveyors, super-heavy service conveyors. Other accessory items include supports, rock ladders, troughing and support idlers, types of belt take-ups, hoppers, etc. Unusual quantity (and quality) of illustrations explain conveyor operation in abundant detail.—Pioneer Engineering, 3200 Como Ave., Minneapolis, Minn.

Concrete pipe handler

A vehicle for in-plant handling of cast concrete pipe up to 6 ft long has been placed in limited production. Clark's Powrworker concrete pipe handler, carries molds for pipe as large as 72 in. inside diameter to casting machines, positions them for casting and transports filled molds to curing areas. Powered by a battery-operated series wound motor with double reduction drive similar to motors of standard pallet trucks, the pipe handler has a 13,000-lb lift capacity. Wheelbase is 80¾ in. Complete specifications available.—Clark Equipment Co., Industrial Truck Div., Battle Creek, Mich.

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